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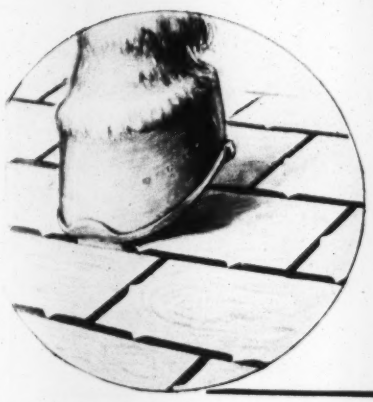
# Kreolite Lug Wood Blocks

"The Paving that's Saving"

**Hauling 16 tons over a Lug Wood Block Pavement**

**"I was sure the horses would get a good foothold on the Lug Wood Blocks,"**



said the teamster of the McGettigan Cartage & Storage Co. of Toledo, Ohio, when asked why he drove his truck and its 16-ton load over the Kreolite Lug Wood Block Pavements on Jefferson Avenue, Toledo, Ohio.

The expansion spaces provided by the side lugs on Kreolite Blocks give a good foothold for horses. The calks on their shoes grip the edges of the blocks, absolutely eliminating all slipperiness of the pavement.

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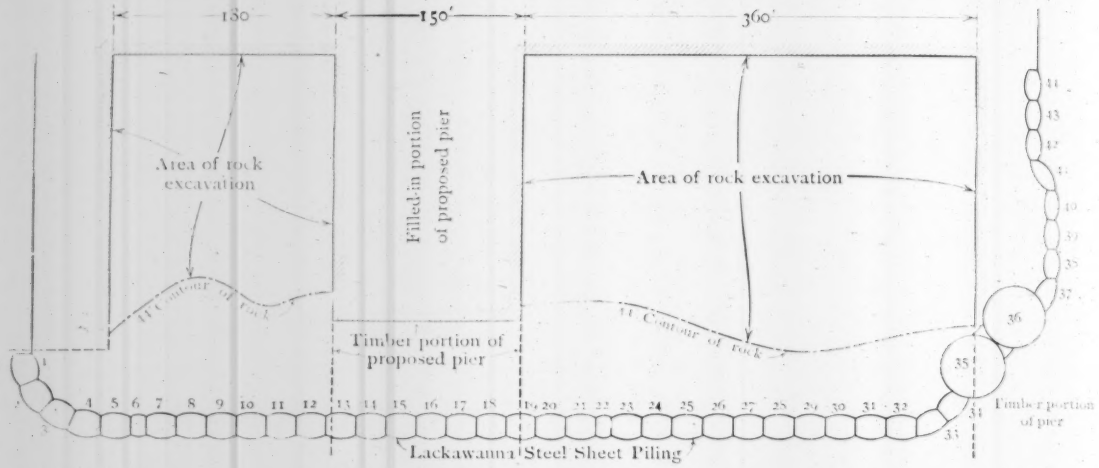
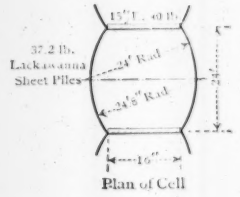



# The Novel Type of Steel Sheet Piling Pocket Utilized in Building the 46th St. Cofferdam, N. Y.

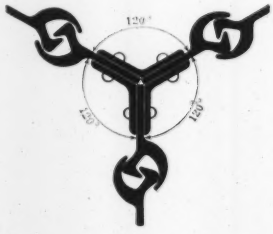
The most nearly comparable precedents for the 46th Street Cofferdam were the circular pocket cofferdam constructed for raising the U. S. Battleship Maine from Havana Harbor and the square pocket cofferdam for the construction of the Black Rock Ship Lock, Buffalo, N. Y. Both of these cofferdams (described in detail in our Bulletins Nos. 102 and 103, which will be sent on application) were constructed exclusively from Lackawanna Steel Sheet Piling, were of the filled pocket type and were designed to be independently stable.



Plan of Cell

The type of pocket used at 46th Street was designed to combine the best features of both circular and square pockets as demonstrated in the Maine and Black Rock works.

Each consisted of two slightly rounded outer walls with joining transverse straight walls 24 feet apart, as shown in the smaller sketch. The convex inner and outer faces varied from 16 feet apart at the transverse walls to 22 feet apart at the centers. The transverse walls were each braced by a heavy channel crossing the wall and bolted to each separate sheet pile, the additional bracing being placed for added stability during construction of the pockets. The connection of one cell to another is made with our special three-way sheet pile known as No. 9. It is evident that this type of pocket has a much greater minimum thickness than a circular pocket and is less subject to distortion than is either a circular or a square pocket.




Two of the cells, numbers 35 and 36, had to be made circular in plan and of much larger size (55 feet diameter) than the standard, owing to the necessity of clearing the corner of an existing pier at the foot of 44th Street.

The two cells, numbers 6 and 22, which were left open up to the time of unwatering, for the passage of barges loaded with construction material, were made narrower than the standard cells, but of the same general type.

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Engineering News

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# Brazilian Opportunities and the Brazil Railway

By F. E. Lawrence*

*SYNOPSIS—Facts about the geography and climate of Brazil, bearing on engineering work. Lines and organization of the Brazil Railway Co. Features of location, construction, maintenance, equipment and operation. Labor conditions. Inducements offered settlers. American engineers warned not to rush into the country without knowledge of language and people—unless employed by a moneyed corporation.*

South America is about as well known to the people of the United States as is central Africa, but our manufacturers are waking up to the surprisingly good opportunities there for trade.

Brazil is greater in area than the United States, excluding Alaska, and is very much larger than Argentina. With double the population of Argentina it has one-half the mileage of railroads; only a fringe along the coast of Brazil is settled. The vast inland country, with its rich soil, great water courses and potential water powers, its trackless forests of hardwoods and pine, remains untouched by civilization. This section of Brazil will experience a wonderful growth in the next decade.

The popular impression of Brazil is that of a hot unhealthful tropical country devoted entirely to coffee and rubber plantations. This conception is far from the truth, for only in the extreme north, in the basin of the great Amazon River, is rubber obtained; and practically all the coffee produced in Brazil, which is, by the way, about 70% of the world's production, comes from the one state of Sao Paulo.

North of the twentieth degree of south latitude, which crosses Brazil consideraly to the north of Rio de Janeiro, the climate is tropical. South of that line the climate is delightful, with the exception of the narrow low coastal strip that lies between the sea and the range of mountains that parallels the east coast. In summer this strip is tropical, but in winter it corresponds to Florida.

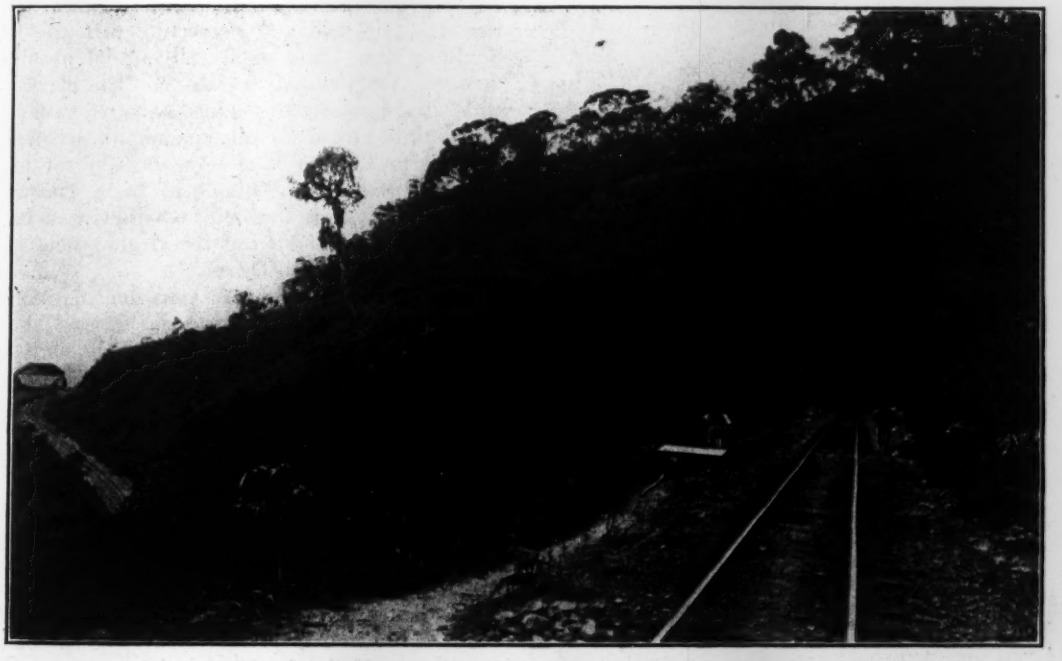


FIG. 1. TYPICAL PIECE OF DEVELOPMENT AND STEEP GRADE ON THE MOUNTAIN DIVISION OF THE PARANA RY., BRAZIL

The Brazil Ry. traverses the four southernmost states of Brazil. Starting at Sao Paulo, the capital of that state, it extends southward to the borders of Uruguay. It comprises practically all the railway lines south of the

*415 West 36th St., Savannah, Ga.

City of Sao Paulo and provides an all-rail route between Rio de Janeiro and Montevideo, the capital of Uruguay, and to Buenos Aires in Argentina.

The Brazil Railway Co. is in effect a holding company, incorporated under the laws of Maine. The physical creation consisted of connecting various existing properties. The organizer and head of the syndicate is a well-known American railroad financier, Percival Farquhar, but the capital employed is largely French, Belgian and English. The central offices, previous to the European War, were in Paris, with branch offices in London and New York. The operation of the properties in Brazil has been, for the most part, in American hands. The war has forced the system to rely, rather prematurely, upon its own efforts at earnings; and despite the exceedingly hard times in Brazil, it is making slow but satisfactory headway. The general direction of the Brazilian Ry. is parallel to the coast, but with one exception (projected) the lines do not operate for coastwise traffic, it being impossible to meet the competition of sea routes.

All the lines are of meter gage—the prevailing standard in Brazil—although some of the English lines and the Government-owned Brazil Central Ry. are broad gage. The adoption of the meter gage was due largely to French influence; French capital initiated railways in southern Brazil, and Brazilian engineers adhere principally to French practice.

### Lines Under Construction and Proposed

The various roads that make up the Brazil Ry. system are as follows:

| Road | Km. in Operation | Km. Under Construction | Total |
|---|---|---|---|
| Sorocabana Ry. | 1,319 | ... | 1,310 |
| Sao Paulo Rio Grande Ry. | 1,439 | 200 | 1,639 |
| Parana Ry. | 417 | ... | 417 |
| North Parana Ry. | 45 | ... | 45 |
| Auxiliaire Ry. | 2,159 | 12 | 2,171 |
| Thereza Christina Ry. | 116 | ... | 116 |
| Sao Paulo Southern Ry. | ... | 140 | 140 |
| Total | 5,486 | 352 | 5,838 |

This is equivalent to 3,628 mi. of track.

Only two of the projected extensions of the Brazil Ry. are now under construction, one being the Orinhas line from Jaguariahyva to Orinhas, the other being the Sao Paulo Southern, mentioned. The following lines have also been projected:

1. The Guarapuava Line—A branch of the Sao Paulo Rio Grande Ry. from Ponta Grossa going westerly to Guarapuava, in the state of Parana. As it will penetrate a new and rich part of the state, it will probably be the next line to be built. It will ultimately be extended farther west to develop the almost unknown interior.

2. The Short Line, Curityba to Sao Paulo—A line running northeast from Curityba, gradually descending the mountain wall and connecting with the Sao Paulo Southern Ry. This route will cut off about 300 km. between Curityba and Sao Paulo. The maximum gradients will be greatly reduced as compared with the present line or with any of the other mountain divisions.

3. Porto Tibirica Line—An extension of the Sorocabana Ry. from Salto Grande to Porto Tibirica, on the Rio Parana. This line will tap the great Rio Parana and initiate an invasion of the virgin wilderness beyond in the state of Matto Grosso.

4. Rio Iguassu Line—An extension of the Sao Paulo Rio Grande west from Porto da Uniao, following the Iguassu River to its mouth at the Falls of the Iguassu, and a branch from this latter point to the Sete Quedas Falls on the Parana River.

Both of these waterwalls are greater in volume of water and height of fall than Niagara. What a region of industry and activity some day this will be! It awaits the colonist and capitalist.

5. Sao Paulo-Montevideo Coast Line—A coast line from Sao Francisco to Porto Alegre, one part of a coast-line route, for which preliminary surveys have been made, from Santos to Montevideo in Uruguay. While this route, 2,700 km. long, will eventually be completed, it is a dream of the future, for the country could not at present furnish enough local traffic to make the line pay, and it could not compete for through service with steamers between those ports.

6. Sao Paulo-Santos Line—At present there is only one line between Sao Paulo and Santos, the Sao Paulo Ry., owned and operated by an English company. Over the mountain it maintains an 8% grade, operated by cables. It handles nearly the entire coffee output of Brazil and is one of the richest short railroads in the world. Capitalized at $200,000 per mi., its annual operating income is close to $100,000 per mi.

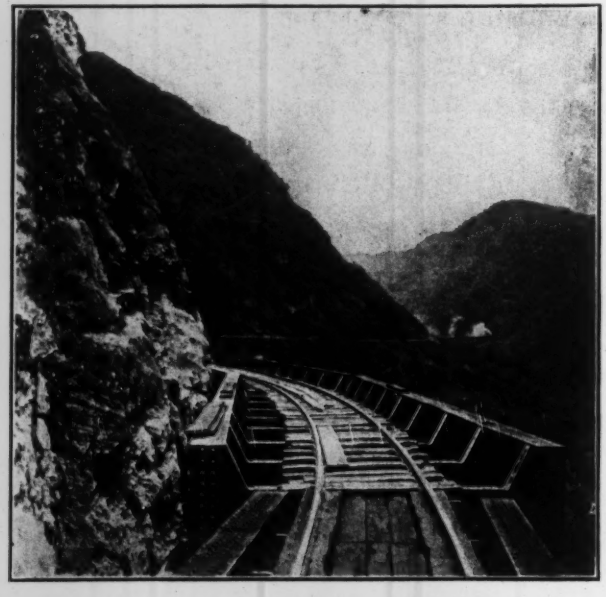


FIG. 2. LINE OF PARANA RY. ASCENDING THE PARANA SERRA

FIG. 3. PLATE GIRDER VIADUCTS; MOUNTAIN DIVISION, PARANA RY.

Studies have been made by S. F. Shaw, former chief engineer of the Brazil Ry., for another line with a grade of 2.5%, which would entail very heavy work, and also for an alternate line with steeper grades. Present conditions have relegated the idea to the background.

The two most interesting "serras," or mountain divisions, are those of the Parana and Sao Francisco Lines. The former was constructed about 30 years ago by the French, and the latter was completed only last year by the Brazil Ry.

All the serras are now operated with Mallet locomotives, most of which weigh, without tender, 78 tons and have a tractive effort of 29,500 lb. Somewhat heavier locomotives are used on the Sorocabana Ry. They give satisfactory results and work well on the sharp curves.

The maximum gradient of the Sao Francisco serra, uncompensated for curvature, is 3%, and the minimum radius of curvature is 100 m. As the line back of this serra has no adverse grades in either direction greater than 0.5%, it provides a low-grade line for all products going out of the country.

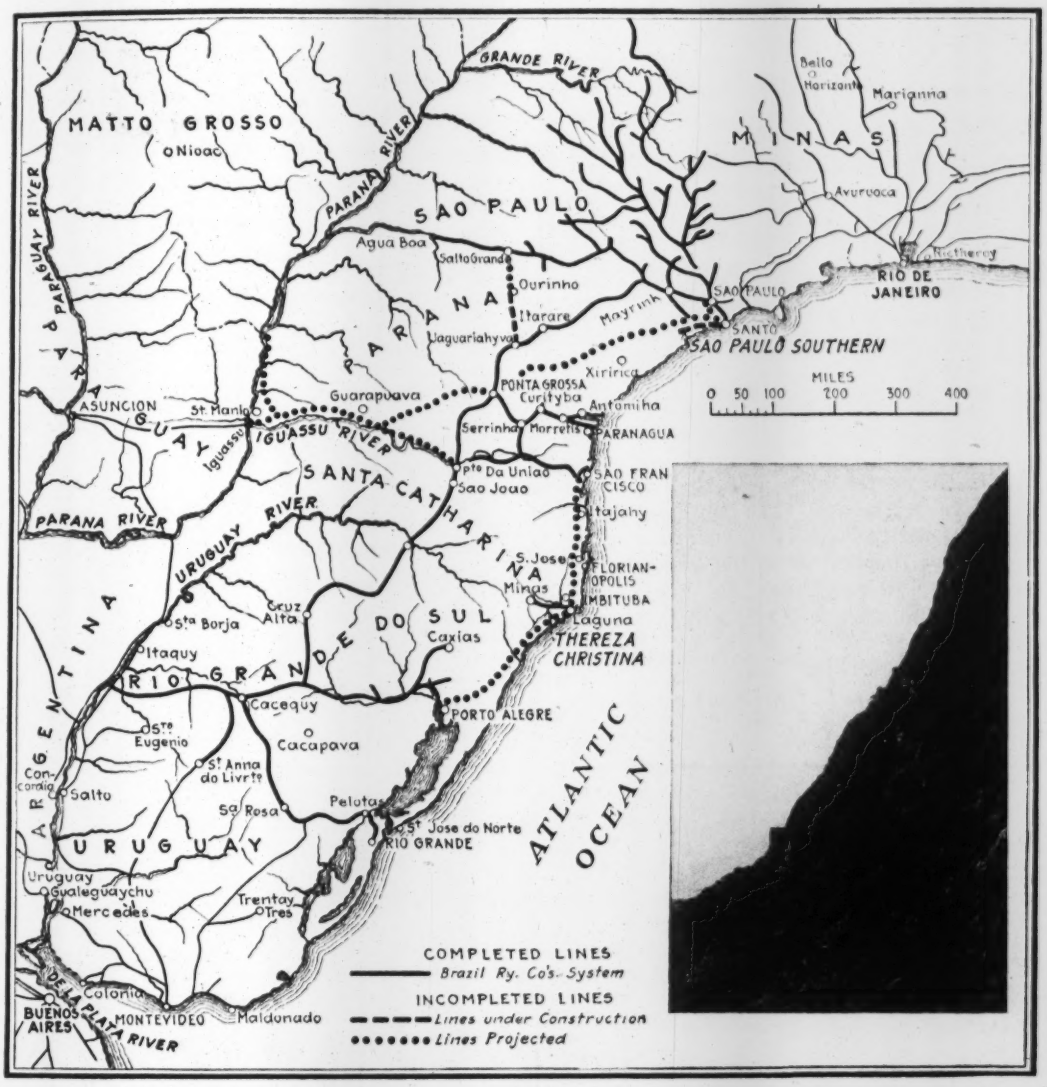



FIG. 4. MAP OF BRAZIL RAILWAY CO'S SYSTEM; VIEW OF VIADUCT ON PARANA SERRA

The engineering features of these two serras are much alike, the track in both places zigzagging up the steep mountain sides, taking advantage of every natural topographical help to gain elevation. The height reached by the Parana serra is 3,123 ft., and this in a distance of 38 km. The maximum grade, compensated for curvature, is nearly 4%, and the minimum radius of curvature is 78 m., which is equivalent to a curvature of approximately 22° 30′ per 100-ft. station. There are in the 38 km. of climb 14 tunnels and a great number of viaducts and bridges.

## Construction and Maintenance

The national Government, through its "fiscal engineers," exercises an authority over the railroads something similar to our Interstate Commerce Commission. All methods and changes of construction and operation must be passed on and approved by the fiscal engineers before they can be put in force. In the construction of bridges, building, culverts and track only the most enduring and permanent types are permissible. Timber

bridges, trestles and culverts are never used, except for temporary use as falsework or until the permanent structure can be erected. The majority of stations are also of masonry. Bridges are constructed of steel and masonry or concrete, and nearly all culverts are of masonry. In Brazil, where cement must be imported, granite masonry with cement or lime-mortar joints is considerably cheaper than concrete, and this type of construction is consequently much more common than concrete.

No attention was paid until recently to the alignment and grades of railways, and as a consequence the older lines of the system are simply a succession of sharp grades with excessive curvature. On many lines no attempt was made to follow the natural water grade. The natural topography of southern Brazil is very broken. Behind the coastal range there are no continuous ridges, but the hills rise up like a field full of immense ant-hills. Ridge routes are therefore, as a rule, out of the question.

Brazil has suffered, as did the United States, from irresponsible railway promoters, whose only object was to get as many kilometers of line, of as light a character as possible, between two points; who made their profit on the construction, with not a thought of the future operation of the line. Many roads are now struggling with the effects of that criminal policy. Of late years there has come a change, and the locations of recently constructed lines compare favorably with the best of those in Europe and North America. The Sao Francisco Line, for example, was partly located by American engineers and constructed by French. With the exception of the mountain division between Hansa and Sao Bento the maximum grade both ways is 0.5%, and the minimum radius of curvature is 150 m., or 70° 40′ metric, and this minimum is seldom used.

Surveys have been made to reconstruct almost the entire Parana Ry. and much of the Sao Paulo Rio Grande

FIG. 5. GREAT CATARACT OF THE RIO IGUASSU

The volume and height of fall exceed that at Niagara. On the western boundary of Parana, hundreds of miles from a railway

and Auxiliaire Railways. The Sorocabana, as a whole, is much better built as respects alignment and grades than any of the other lines.

The T-rail, as used in the United States, but of European design, has been in use on all the lines from their inception, the weights used averaging from 40 to 60 lb. per yd. In 1912 a great many thousands of tons of new rail—75 lb. per yd., Am. Soc. C. E. section—was ordered in France and Belgium to relay most of the Sorocabana, Parana and Auxiliaire Railways. A considerable amount of this rail has been laid, but many hundred kilometers yet remain to be put down.

Practically all sleepers are of hardwood, in which it is first necessary to bore holes before spikes can be driven. On most of the new lines screw spikes are used. The timber used for sleepers (principally "Imbuia," of a dark

FIG. 6. TERMINAL OF THE PARANA RY. AT PARANAGUA

Locomotive fuel briquets piled ready for loading

greenish color, which makes beautiful furniture) lasts on an average from 16 to 20 yr.; and if free from faults when placed in the track, it will last for 35 yr.

Tie-plates or rail-braces are rarely used, for even on the mountain grades, with their very sharp curves and heavy locomotives, no trouble is incurred with canting rails, as the timber is hard enough to resist the cutting action, and spikes once driven do not come out.

The usage with respect to ballast, both rock and earth, is to cover the track completely, flush with the bottom of the head of the rail. It is claimed that the timber in the track lasts longer and that the track on the sharp curves is held in place better by this method. On the other hand, it is very difficult to inspect the condition of the sleepers and rail connections. This mode of ballasting certainly reduces the noise in passenger travel, the principal noise noticed being the gritty grinding around curves.

On the older parts of the roads the European practice of placing the joints opposite each other is used, which causes an annoying series of jolts on track that is not kept in perfect condition. The joints on all recently built lines are staggered, and wherever new rail is laid.

### Rolling Stock and Fuel

Since the beginning of operation the majority of the locomotives used have been of American make, supplied almost exclusively by the Baldwin Locomotive Works. The Auxiliaire Ry. had besides a number of German locomotives.

After the different roads had been incorporated into the Brazil Ry., Mallet locomotives—type 2-6-6-2, tractive effort 29,500 lb.—were purchased for all the mountain divisions; and a number of type 0-6-4, tractive effort 19,000 lb., were secured for passenger and freight service in the plateau country. Practically all these locomotives, although designed by American mechanical engineers and according to American standards, were purchased from the Henchel Locomotive Works, of Cassel, Germany.

Subsequent orders were placed with the American Locomotive Co. for a number of similar locomotives, but equipped with superheaters and other modern appliances. These latter Mallets have shown to great advantage as

compared with the first, for they handle from 10 to 20% more load and use about 20% less fuel.

The rolling stock, box- and flat-cars, etc., used on the various lines were all of European pattern and build and were, for the most part, 4-wheel, 4-, 6-, and 12-ton wagons with link couplings. The passenger coaches also were of antique 4- to 8-wheel design and weighed about 9 tons. Since the Brazil Railway Co. assumed control, box-, cattle- and flat-cars of 30 and 40 tons' capacity, and very comfortable modern passenger coaches, wicker seated and electric lighted, were purchased in Belgium, having also been designed according to the latest American specifications. Automatic couplers and vacuum brakes were introduced on all new rolling stock, both passenger and freight.

The Sorocabana and Auxiliaire Railways use coal exclusively for fuel, and the Parana and Sao Paulo Rio Grande Railways use coal for passenger and wood for freight trains, although on the mountain divisions some coal is used in conjuction with wood by freight trains.

The coal is, or was previous to the war, purchased in the British Isles and comes in briquettes. It is shipped to the various ports, unloaded by hand from the hold of the vessel, stored or loaded again for transportation over the line to intermediate points, stored at these points and finally loaded on the tender of the engine. All this transferring is done by hand, the method of loading or unloading being a human chain passing the briquettes one by one. Each briquette must pass through from 30 to 50 men's hands from the time that it is unloaded at the dock to the time it enters the locomotive firebox. When the coal is piled for storage, it is arranged in geometrical pyramids, carefully built up.

The cost of coal on the tender averages about $16 per ton, which makes the fuel item of operating expenses very high. On the Parana Ry. the fuel item for 1913, during which time coal was used exclusively, amounted to 31% of the total operating expenses.

The use of wood for fuel on those parts of the system where wood abounds brings the charge down somewhat, but at the usual price for wood not much saving is obtained. It requires 8 cu.m. of wood to furnish the same number of heat units as 1 ton of coal, and the wood costs ordinarily about $1.50 per m. on the tender. The service obtained with wood, however, is far inferior to that with coal, and on long hard climbs it is impossible to keep steam pressure up to maximum-load requirements. It has been determined by tests with engines now in use that, with wood fuel, the rating for any engine for any ruling grade should be fixed at 90% of the rating when using coal. The locomotives are, of course, adapted to use either coal or wood.

Up to the present time no coal of a quality fit for use has been discovered in Brazil. On the other hand, the country abounds with hydro-electric-power possibilities. It is probable that good coal will ultimately be discovered, and oil may be introduced; but the real solution of fuel problems is the development of the available water powers.

The Sao Paulo Ry. is shortly to electrify its mountain division between Sao Paulo and Santos, and the Parana Ry. will ultimately electrify its serra, to comply with its contract with the Government. These mountain divisions are the logical starting points from which this work will spread to the other parts of the system.

The "chief engineers of the line," "chief engineers of motive power" and some of the other divisional chiefs of the different roads that make up the Brazil Ry. are at present mostly Germans who have grown up with the roads. They immigrated years ago to Brazil, engaged in the construction and have, with characteristic perseverance and steadiness, remained with the roads.

The general officials, superintendents of the various roads and the engineers have been mostly foreigners, principally Americans, and most of the present higher officials are of the latter nationality.

### Railway-Labor Conditions

There are no labor unions in Brazil. The great majority of enginemen and a considerable number of conductors are of foreign extraction, the enginemen being principally German, Italian and French, with some Austrian and a few English. There are, of course, Brazilian enginemen, but on the lines south of Sao Paulo they are in the minority. The pay of enginemen is from 200 to 300 milreis per month, which amounts under the normal rate of exchange to an average of about $80. Houses are usually provided.

Conductors are mainly native Brazilian, although there is a sprinkling of foreign born also. The average pay for a conductor is about $60 per month, and of flagmen, brakemen, etc., from $30 to $40.

There are no "16-hr." laws or any limitations of time spent on the road; but there are laws that there shall be a switchman at every main-line switch stand as the train passes, and the number of brakemen per train is greater than here, as a result, no doubt, of the exclusive use until recently of hand brakes.

The staff system of train orders and operation is in use, blocks being established from station to station, with the station agent acting as dispatcher for his block. This will undoubtedly be changed as soon as traffic warrants.

Track labor is composed of both natives and immigrants; all nations are represented. Day labor receives normally about $1 per day, section houses and firewood being provided. Section foremen receive about $40 per month, and the position that corresponds to supervisor of track in the United States pays $90 to $100, houses being in all cases provided by the company.

The general rule with respect to the amount of track labor employed is 1 man per km. of track, and the Government engineers usually insist upon this standard.

The sections, as a rule, are from 6 to 8 km. long, and sometimes 10 km., short sections being favored, as neither hand-power nor motor cars are used, the hand push-car being used exclusively by section gangs.

### Land and Colonization Department

As part of the concessions of the Brazil Ry. granted by the Government, immense tracts of land in various states were turned over to the railway, some of which the lines run through and some of which are hundreds of kilometers from any railroad.

Such blocks of land have been conceded in Matto Grosso, Parana, Santa Catharina and Rio Grande do Sul and are used by the company for colonization and agricultural experimental purposes. Some are being developed by certain subsidiary companies mentioned later.

The railway has a well-organized land and colonization department, which devotes its energies to obtaining set-

tlers and looking after them and developing the lands of the company. The Government has also established a number of colonies in various states.

Unusual inducements are made by both the Government and the railway to immigrants who bring their families and take up land. Their ocean transportation is, in most cases, paid; they are carried over the railroads free of charge, with their tools and effects, from the seacoast to wherever they intend to settle and are assisted or conducted by a representative of the railway or Government to their destination. They are then given a lot or parcel of land, which is to be paid for in one year or two years or more, as the case may be; and enough tools and provisions are advanced them to carry them over their first season.

The Brazil Railway Co. has initiated several subsidiary enterprises of large extent. The Brazil Lumber and Colonization Co., capitalized at about $5,000,000, has been cutting and sawing lumber in the states of Parana and Santa Catharina, where it has one very large modern plant and two smaller ones. It cuts principally Parana pine and some of the hardwoods. This company forms colonies to settle up the land on which the timber has been cut and has established a settlement at Tres Barras.

The Brazil Land and Cattle Co. is utilizing great tracts of land in the states of Parana, Sao Paulo and Matto Grosso for cattle raising. It has introduced blooded stock from the United States to form a nucleus upon which to build up a line of better cattle than that native to Brazil. The company is, in addition, interested in a packing plant in Sao Paulo, which is being erected by one of the large American packing houses.

### Warning to American Engineers

Finally, a word to American engineers who desire to try their hand in South America: I am firmly convinced that South America from Sao Paulo south will experience a wonderful growth in the next few years. It is a marvelous country in many ways, undeveloped and abounding in opportunity. The freedom of its plains and the immensity of its wooded wilds inspire in one a desire to work and build. I suppose it is the instinctive spirit of the pioneer on the frontiers of civilization. And yet it is a mistake to rush off to it, unprepared in finances or language or knowledge of the ways of living there.

With a capital of a few thousand dollars a man can go there and pay for his experience while laying the foundation of his business; but to the engineer who is looking for a job my advice is strongly against going there expecting to find it. The good salaries go only to those who are sent there, to build and operate industries, by outside interests that are willing to pay for the man's first year of experience; for to the company employing him almost any man is practically a dead loss for the first year, or until he can acquire a knowledge of the language, the people and their ways of doing business. Certainly the most helpless of mortals is a person stranded in a strange land, unable to speak the language and knowing nothing of the people around him. Much useful knowledge can be gained at home by close study.

The moneyed interests of this country are becoming more and more impressed with possibilities in South America. These corporations are the means by which American engineers may fulfill their desire for a South American career.

# Mississippi River Stages, 1915, and Hydrograph, 1861-1914

By James W. Skelly*

The stages of the Mississippi River at St. Louis during the summer of 1915, and January, 1916, were remarkable in many respects, particularly in the frequency and long duration of high stages. While not equaling the extremes of former years, notably the extreme high stage (38 ft.) of June 10, 1903, or the more famous flood stage (41.3 ft.) of June 27, 1844, there is nevertheless presented the very unusual and unprecedented occurrence of five distinct rises within three months—two in June, one in July, two in August—and a sixth in January, 1916, all of which slightly exceeded the "bank full" stage (30 ft.).

The crest on Aug. 22 (see accompanying hydrograph) followed the widespread rainfall of the Gulf storm that wrought so much havoc at Galveston. The crest of January was the result of heavy local rains augmented by a flood in the Illinois River, a tributary, and is remarkable, not only for its occurrence at a time when the Mississippi is usually very low, but because of the limited upstream range of its influence.

Before considering the diagram a few words may be said regarding the gage records for the period from Jan. 1 to Feb. 10, 1916, which have become available since the diagram was plotted. These figures are as follows:

| Date | Reading | Date | Reading |
|---|---|---|---|
| Jan. 1 | 4.0 | Jan. 22 | 7.7 |
| 2 | 5.7 | 23 | 13.5 |
| 3 | 11.0 | 24 | 20.5 |
| 4 | 15.2 | 25 | 23.8 |
| 5 | 15.6 | 26 | 23.8 |
| 6 | 14.9 | 27 | 23.7 |
| 7 | 13.8 | 28 | 26.1 |
| 8 | 12.3 | 29 | 28.0 |
| 9 | 11.0 | 30 | 30.0 |
| 10 | 10.6 | 31 | 31.4 |
| 11 | 10.2 | Feb. 1 | 31.3 |
| 12 | 11.2 | 2 | 30.8 |
| 13 | 15.8 | 3 | 29.7 |
| 14 | 18.4 | 4 | 27.8 |
| 15 | 17.4 | 5 | 25.8 |
| 16 | 15.4 | 6 | 23.5 |
| 17 | 14.0 | 7 | 21.1 |
| 18 | 12.2 | 8 | 18.7 |
| 19 | 9.1 | 9 | 16.4 |
| 20 | 7.3 | 10 | 15.0 |
| 21 | 6.8 | | |

At the crest, Jan. 31, 1916, the Missouri River, the principal tributary above St. Louis, was at mean or ordinary stage at Lexington, Mo., only 340 mi. above St. Louis, and was frozen over at St. Joseph, 500 mi. above; the upper Mississippi River at Dubuque, Iowa, 430 mi. above, was also at mean stage and frozen. The rises and crests of July, August and January established new high records at St. Louis for almost the entire periods July 22 to Aug. 29 and Jan. 27 to Feb. 8.

The accompanying comprehensive hydrograph, showing the daily stages at St. Louis for 1915, shows also the extremes for all years from 1861 to 1915, including, in addition, the famous flood of 1844 and the lesser one of 1858, the courses of both of which above the 30-ft. stage are shown because of their historic interest in the middle Mississippi Valley. The flood of 1858 (37 ft. on the present St. Louis gage), although lower at St. Louis than the 1903 flood, was higher at other near-by places both above and below that city, the greater local height of the 1903 flood being due to the contraction of the waterway by the construction of levees and other similar embankments in the vicinity of St. Louis.

*United States Assistant Engineer, Custom House, St. Louis

The extreme high and low daily stages shown on the hydrograph are the extremes for each day in the year; and the mean high and low daily stages, 25.9 and 3.9 ft. respectively, are the means of the foregoing extremes—365 in each case. The mean high and low yearly stages, 26.9 and 2.2 ft. respectively, are the means of their respective extremes for each day throughout the period covered by the hydrograph—54 years. The mean stage of river, 12.4 ft., is the mean of all daily stages. The curve of mean daily stages shows the mean for each day in the year, and the high and low normal limits indicate means in their respective zones. There is also shown the curve of river discharge at St. Louis, derived from about 500 observations, covering a period of 50 years.

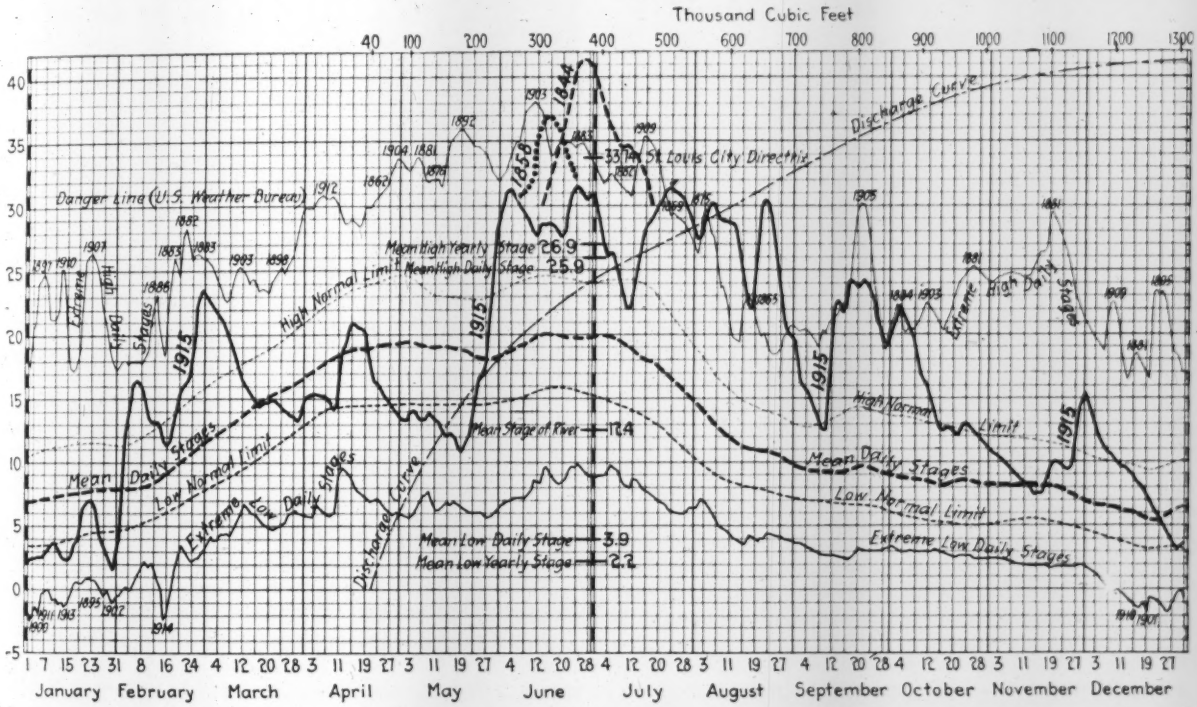
## Fusible Plugs Investigated by Bureau of Standards

Fusible plugs in steam boilers are usually screwed through the steel heating surface of the boiler an inch or so above the line at which low water in the boiler begins to be dangerous. So long as the plug is covered with water it remains intact; but when the water level falls below the plug, the metal is melted by the heat of the fire, and the steam blowing through the opening gives warning of the dangerous condition.

While fusible plugs have been in widely extended use since the middle of the last century, they have often failed to operate in the manner intended and have consequently become more or less discredited by engineers. Their use is required on the boilers of all American merchant vessels; but they are not used in the United States Navy, nor are they generally used in marine boilers abroad. They are extensively used in this country in locomotive and stationary boilers and in portable boilers.

The Bureau of Standards has recently conducted an exhaustive investigation into the causes of the failure of fusible-tin boiler plugs, and a report on the investigation has been issued as No. 53 of the bureau's technologic papers. The investigators found that the cause of the failure of fusible-tin boiler plugs has generally been the presence of lead and zinc as impurities in the tin. Even very small percentages of lead and zinc impurities may cause the plug to fail to operate as it should after some months of exposure to heat and oxidizing influences. The bureau recommends that specifications for fusible plugs should require the tin to be at least 99.7% pure and to contain not more than 0.1% of lead and 0.1% of zinc. Even purer tin than this is obtainable in the market and might well be specified for such plugs.

Comparing these requirements with present standard specifications, it is of interest to observe that the specification of the Boiler Inspection Department of the Interstate Commerce Commission totally ignores the purity of the tin. The specification contained in the standard boiler code recently adopted by the American Society of Mechanical Engineers calls for tin that will melt between 400° and 500°; but this specification is of no value, since it will admit tin containing as much as 65% of lead or 15% of zinc. The Federal Steamboat Inspection Service is to revise its specification to conform to the recommendations of the Bureau of Standards.


Thousand Cubic Feet
Discharge Curve
St. Louis City Directrix
Danger Line (U.S. Weather Bureau)
Mean High Yearly Stage 26.9
Mean High Daily Stage 25.9
Mean Stage of River 12.4
Mean Low Daily Stage 3.9
Mean Low Yearly Stage 2.2
Mean Daily Stages
High Normal Limit
Low Normal Limit
Extreme High Daily Stages
Extreme Low Daily Stages
January February March April May June July August September October November December

MISSISSIPPI RIVER HYDROGRAPH, ST. LOUIS GAGE, 1861 TO 1914, AND STAGES FOR 1915

**The Tansa Dam,** which impounds water for the supply of Bombay, India, has been raised from a height of 118 ft. to 126½ ft. as a part of a project to increase the water-supply. The enlarged works were dedicated on Dec. 8, 1915. The dam is of masonry and had an original length of 1½ mi. By adding 8½ ft. to its height its impounding capacity was increased from 21,600,000,000 to 32,400,000,000 gal. The raising of the dam was completed in May, 1915. Particulars of the whole project are given in the London "Surveyor" for Jan. 7, 1916.

# Tar-Coated Concrete Pavement in Ann Arbor, Mich.*

By Manley Osgood†

The more or less famous Ann Arbor tarred-concrete pavement is a concrete pavement, usually laid in two courses, with a carpet coating of tar and sand placed as soon as the concrete is sufficiently cured. This type of pavement originated in Ann Arbor in a peculiar way. In 1907, during the construction of a new City Hall, a barrel of tar was accidentally spilled over a concrete walk. Sand was thrown into the tar to make the walk fit for foot traffic. The tar and sand adhered to the concrete, and the carpeting thus formed is intact today. This accident furnished the idea for the carpeted pavement.

The first pavement of this type was constructed on Maynard St. in 1909. It is one block long and contains 1,880 sq.yd. The materials were purchased, the work of construction was done by the city, and every precaution was taken to insure good results with the materials used. This pavement was laid for $1.03 per sq.yd., including curbing and grading, or for 78c. per sq.yd. for the pavement only.

When the news spread that this pavement had been constructed so cheaply, the petitions commenced pouring into the Common Council from all parts of the city; and all expressed a preference for "pavement similar to that on Maynard St." The result was that the city started paving streets as fast as was possible with the machinery and labor available. This can best be illustrated by the yardages laid during the succeeding years:

| | Sq.Yd. | | Sq.Yd. |
|---|---|---|---|
| 1909 | 1,880 | 1912 | 11,540 |
| 1910 | 16,560 | 1913 | 33,460 |
| 1911 | 64,090 | 1914 | 11,630 |

The drop in 1912 was due principally to water-works agitation which developed that year and perhaps secondarily to the fact that a change was made in the City Engineer's Office in 1912. I assumed the office of City Engineer on June 17, 1912, coming from an Ohio city and having no assistants who held over from the previous administration. The drop in 1914 was due to a charter clause which provides that not more than $150,000 worth of paving bonds shall be outstanding at any time. This limit was reached in 1914, and now only so much paving may be done each year as will require a bond issue equal to those redeemed the previous year.

## Attempts to Cheapen the First Cost

In 1910 two things happened that worked seriously to the detriment of the pavement laid in that year and in 1911 (which constitutes about 58% of all the concrete pavement now laid in the city). Bids were called for and received; but the City Council, on the advice of its officers, rejected all bids and ordered these officers to proceed with the construction on a force-account basis. The specifications under which bids were received called for a 5-in. lower course and a 2-in. top course of concrete. In order to insure a substantial saving in cost over the lowest bid received, the lower course was made 4½ in. thick and the top course 1½ in. thick, a total thickness of 6 in. instead of 7 in.

*From a paper presented at the annual meeting of the Michigan Engineering Society.

†City Engineer, Ann Arbor, Mich.

In 1910 also a patent was applied for, a company organized, and this type of pavement was advertised and promoted throughout the country. Low first cost was the most advertised feature, and the results obtained at Ann Arbor were used as examples of remarkable savings. The application for a patent (which was never granted) was made by a city employee with much authority in paving construction. As he received a royalty on every yard of this type of pavement licensed or constructed by the company promoting it, he was very materially interested.

In these two years, 1910 and 1911, everything was sacrificed to first cost. The small batch concrete mixer used on Maynard St. was replaced by continuous mixers of greater capacity. Cement was never tested, unless there was some reason to suppose that it did not come up to specifications. Bank-run gravel was used in both courses without screening or washing; and especially in 1911, when the greatest yardage was laid, there was not sufficient, or sufficiently competent, supervision or inspection to insure even the best obtainable bank-run gravel.

Many who are familiar with the paving operation that year have told of seeing lumps of clay or even of sod in the gravel, as it lay ready to be shoveled into the mixer. On one section, three blocks long, of a comparatively heavily traveled street with a car track in the center, a further saving was effected by building a one-course concrete pavement of 1 part cement to 5 parts of this same bank-run gravel. After the pavement was entirely completed, the number of sacks of cement used was checked against the yardage, and it was found that not sufficient cement had been used to make even a 1:5 mixture. I may say here that, although this section of pavement was constructed no longer ago than 1911, the city is now facing the proposition of surfacing it with some stable material or reconstructing it. The carpet coating will not adhere to this lean mixture.

No reinforcing of any type was placed in these concrete pavements, nor were the expansion joints protected in any way. Care was not taken to see that these joints were opened and clean to the full depth of the pavement, but sweepings from the concrete prior to carpeting were brushed into them and the remainder of the joint filled with tar and sand poured separately.

## Experiments to Improve the Pavement

In 1912 the thickness of the pavement was increased to 7 in., with a 5-in. lower course and a 2-in. upper course, and care was taken to insure the best grade of bank gravel obtainable. In 1913 we commenced using a washed and screened sand in the top course, retaining the bank gravel for the lower course. Care was also taken to have an open expansion joint the full depth of the concrete, and these joints were filled with filler asphalt.

Many experiments were made to determine the best bituminous material for the carpet coating. Bermudez asphalt was tried, but in the winter time it became brittle and scaled from the concrete. All grades of coal tar were tested—from the crude product to that with considerable refining. The results indicated that the coal-tar product with only sufficient refining to drive off the water and lighter oils was best adapted to the conditions in Ann Arbor. The crude tar was purchased at the local gas plant and sufficient refining given it by heating to about 220° to 250° F., as it was applied to the concrete. During

1911 a bitumen was purchased from the Barrett Manufacturing Co., with about the same amount of refining as that company's lightest grade of Tarvia, but it showed no better results than the local tar refined.

Various methods of application were tried. At one time the only sweeping was done with a rotary broom. This was followed by a gravity tar spreader, which in turn was followed by an automatic sand distributor. At another time the pavement was swept by hand, the tar applied with pouring pots and squeegees and the sand spread with shovels. Various combinations of hand and machine work were used. The tar and sand were even applied to the concrete surface after sprinkling with water. The results obtained indicate that the surface to be carpeted should be thoroughly dry and free from dust and that the tar should be applied sufficiently hot to be in a thoroughly liquid state. Whether the tar and sand are applied by machine or by hand does not affect its lasting qualities so much as does the temperature of the concrete to which it is applied. The carpeting should be applied during hot, dry weather. A one-size gravel

RECENT PAVEMENTS IN ANN ARBOR

| District No. | When Laid | No. Sq.Yd. | Total Cost | Total Cost per Sq.Yd. | Cost per Sq. Yd., Pavement Only | Kind |
|---|---|---|---|---|---|---|
| 1-2 | 1898 | 12,730 | $31,375 | $2.46 | $1.64 | Brick on concrete |
| 3 | 1899 | 5,140 | 11,645 | 2.27 | 1.73 | Brick on concrete |
| 4 | 1900 | 10,791 | 27,845 | 2.58 | 2.17 | Asphalt block on sand |
| 5 | 1902 | 12,484 | 31,778 | 2.55 | 2.16 | Asphalt block on sand |
| 6 | 1902 | 1,589 | 2,860 | 1.80 | 1.39 | Brick on sand |
| 7 | 1903 | 9,140 | 24,486 | 2.68 | 2.19 | Asphalt block on sand |
| 8 | 1903 | 2,431 | 6,490 | 2.67 | 2.19 | Asphalt block on sand |
| 9 | 1907 | 7,519 | 19,092 | 2.54 | 2.03 | Bitulithic |
| 10 | 1905 | 5,045 | 10,720 | 2.03 | 1.63 | Brick on concrete |
| 11 | 1908 | 350 | 672 | 1.92 | 1.50 | Brick on concrete |
| 12 | 1909 | 1,883 | 1,930 | 1.03 | .78 | Ann Arbor tar concrete |
| 13 | 1910 | 1,159 | 1,351 | 1.17 | .88 | Ann Arbor tar concrete |
| 14 | 1910 | 6,765 | 6,685 | .99 | .76 | Ann Arbor tar concrete |
| 15 | 1910 | 8,633 | 8,189 | .95 | .71 | Ann Arbor tar concrete |
| 16 | 1911 | 12,092 | 12,392 | 1.03 | .78 | Ann Arbor tar concrete |
| 17 | 1911 | 2,587 | 2,672 | 1.03 | .84 | Ann Arbor tar concrete |
| 18 | 1911 | 20,826 | 23,172 | 1.11 | .78 | Ann Arbor tar concrete |
| 19 | 1911 | 10,117 | 11,110 | 1.10 | .82 | Ann Arbor tar concrete |
| 20 | 1913 | 5,682 | 7,726 | 1.36 | .93 | Ann Arbor tar concrete |
| 21 | 1911 | 10,483 | 11,678 | 1.12 | .84 | Ann Arbor tar concrete |
| 22 | 1911 | 293 | 314 | 1.07 | .84 | Ann Arbor tar concrete |
| 23 | 1911 | 920 | 1,029 | 1.12 | .77 | Ann Arbor tar concrete |
| 24 | 1911 | 1,833 | 1,747 | .95 | .77 | Ann Arbor tar concrete |
| 25 | 1911 | 3,158 | 3,181 | 1.01 | .78 | Ann Arbor tar concrete |
| 26 | 1912 | 2,407 | 2,703 | 1.12 | .89 | Ann Arbor tar concrete |
| 27 | 1912 | 7,711 | 9,071 | 1.18 | .91 | Ann Arbor tar concrete |
| 28 | 1913 | 15,612 | 20,836 | 1.34 | .98 | Ann Arbor tar concrete |
| 29 | 1913 | 14,131 | 19,257 | 1.36 | 1.01 | Ann Arbor tar concrete |
| 30 | 1914 | 11,832 | 14,802 | 1.25 | .98 | Ann Arbor tar concrete |
| 32 | 1915 | 2,085 | 2,740 | 1.33 | ..... | Reinforced concrete |
| 33 | 1915 | 4,924 | 8,665 | 1.62 | ..... | Asphaltic concrete |
| 34 | 1915 | 14,372 | 24,733 | 1.60 | ..... | Asphaltic concrete |
| 35 | 1915 | 3,273 | 5,431 | 1.66 | ..... | Reinforced concrete |

screening, between the 1/8-in. and 1/4-in. screens, gives better results than were obtained with other sands and gravels tried.

The distance between transverse expansion joints was varied from 25 to 50 ft. The longer blocks developed a transverse crack approximately in the center. A spacing of 30 ft. between these joints was found best.

### Property Owners Learn by Experience

As I have stated, after the paving of Maynard St. in 1909 the people of Ann Arbor were very enthusiastic about this type of pavement. It continued to grow in favor during 1910 and 1911. Then the tide commenced very slowly to turn. It became evident to all that it would be necessary to recarpet the concrete every two or three years and in some cases probably every year, varying with traffic conditions. Those who owned automobiles did not like the throwing up of tar and sand onto the under sides of the fenders. Owners of horse-drawn vehicles avoided the newly carpeted streets, and even pedestrians avoided the crossings. In some places where the carpeting peeled, the concrete commenced to ravel; and sections varying in size had to be cut out and replaced with better material. Then the one-course concrete section previously mentioned wore away rapidly, and two other streets laid the same year showed evidence of soon needing extensive repairs.

A majority of the people now realize that reducing the first cost to a minimum, together with insufficient inspection, has imposed a heavy burden on the community in the expense of maintenance, repair and reconstruction.

But this type of pavement did do one real good to the City of Ann Arbor. Prior to 1909 there were only 67,220 sq.yd. of pavement in the city, and property owners could not be persuaded to petition for more. In the two years following the completion of Maynard St. petitions were received for more than 200,000 sq.yd., some of which is not yet constructed. The introduction of this type of pavement—cheap in first cost—hastened the construction of sufficient pavement to make the property owners realize the economic value of the paved street. Now they are still petitioning for pavements, even though they realize that the first cost will be somewhat higher than was the cost of the tarred-concrete pavements. Furthermore, we should not have even the 231,000 yd. of paving that we now have, had it not been for the low first cost of most of it.

### Tar Carpet for Asphalt-Block Pavement

The carpet coating of tar and sand has been put to other use in Ann Arbor than covering concrete. During the years from 1900 to 1903 a total of 32,500 sq.yd. of asphalt-block pavement was laid on some of the most heavily traveled streets in the city, with no other foundation than the compacted gravel soil. As early as 1908 it was realized that some measure must be adopted to preserve this pavement, and it was given such a carpeting. The success of this carpeting strengthened the faith in a carpeted concrete pavement, the idea for which had been conceived the previous year. The tar and sand adhere very well to the asphaltic surface of these block-paved streets, and the use of this carpeting has prolonged the life of these pavements for many years. On these streets one such coating lasts for about four years.

To summarize the advantages and disadvantages of a carpeted concrete pavement laid in the best possible manner: The cost of the coating of tar and sand just about offsets the cost of reinforcing and joint protection that would be necessary if the concrete were not to be protected by the carpeting. This carpeting waterproofs the surface and makes it less absorptive, therefore more sanitary. It also deadens the sound made by horses' hoofs on the unprotected concrete and does away with any sun glare from uncolored concrete. The disadvantages, which are due to the fact that the carpeting does not adhere well to the concrete, are the expense of maintaining and the annoyance of freshly tarred streets every summer.

Ann Arbor has learned its lesson. Concrete pavements, as a class, are not condemned. In fact, about 5,350 sq.yd. of reinforced-concrete pavement was constructed in Ann Arbor during the past summer; but the gravel used was washed and screened and remixed in a batch mixer, and every precaution was taken to insure the best materials and proper construction. The property owners in most cases are now willing to have the type of a proposed pavement decided by sound engineering consideration; but whatever type is constructed, they want only first-class materials and first-class work.

# Bending Moment in Continuous Reinforced-Concrete Beams

By Raymond J. Roark*

In the design of a reinforced-concrete girder continuous over three or more supports, it is customary to solve for bending moments by the ordinary methods of continuous-beam analysis. The moments at points throughout the beam having been determined, the percentage of reinforcement at any section is made sufficient to care for the moment at that section. The amount of steel and hence the moment of inertia of the beam section are therefore not constant, but may vary considerably in each span; and the ordinary theory, based on the assumption of uniform moment of inertia, cannot be regarded as strictly applicable.

While it is possible to make a theoretically exact analysis, in which account is taken of the variation of moment of inertia, this is rarely done, nor indeed is such refinement generally warranted. It is of interest, however, to know how great the probable error due to the neglect of this factor may be. In the following discussion two cases are assumed and the maximum moments computed—first, by the ordinary theory as adapted to beams of uniform section, and second, by a more exact method taking into account the variation in moment of inertia.

A beam of rectangular section and of three equal spans will first be assumed. If the depth be taken constant, the amount of steel at any section determines to a large extent the moment of inertia at that section. Since the amount of steel is proportional to the bending moment, the moment of inertia of the beam section may be said to vary approximately as the bending moment. Now this variation is least for a beam carrying live-load only, and greatest for a beam carrying dead-load only. Therefore if a beam be designed for dead-load only, it will have the greatest possible variation in moment of inertia and so represent the greatest possible departure from the assumed condition.

Such a beam is shown in Fig. 1. The positions of the points of inflection are assumed to be the same as for a beam of uniform section, as this assumption is

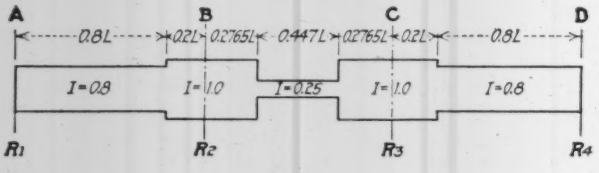



FIG. 1

| Method of Computation | $R_1 = R_4$ | $R_2 = R_3$ | +M in First Span | —M at Supports | +M in Center Span |
|---|---|---|---|---|---|
| Usual method. | 0.400W | 1.10W | 0.0800Wl | 0.10Wl | 0.025Wl |
| Exact method. | 0.389W | 1.111W | 0.07566Wl | 0.111Wl | 0.014Wl |
| % error..... | 2.8 | 1.0 | 5.7 | 10.0 | 78.0 |

made in design, and the moment of inertia is taken as constant between points of inflection and proportional to the maximum moment occurring in that portion of the beam.

The analysis is made by the method of redundant reactions. (A full explanation of this method will be found in Johnson, Bryan and Turneaure's "Modern Framed Structures," Part 2.) Reactions $R_1$ and $R_4$ are taken as redundant, and the deflection at the left end is placed equal to zero. The resulting equation is

$$\int_A^D \frac{Mdx}{EI} m_1 = O$$

where $M$ is the true bending moment at any section and $m_1$ is the bending moment at any section due to an upward load of unity acting at $A$ when the two redundant reactions are removed. $M$ is equal to $M^1 + R_1 m_1 + R_4 m_4$, where $M^1$ is the moment at any section due to the load on the beam with the redundant reactions removed; $m_1$ is as previously defined, and $m_4$ is the moment at any section due to an upward load of unity acting at $D$ when the redundant reactions are removed. Since the beam and the load are symmetrical, $R_4 = R_1$. $M^1$, $m_1$ and $m_4$ can all be expressed in terms of $x$ and the above integration performed.

This integration must be performed first for the distance $A$ to $B$ in two parts, the limits being 0 and $0.8L$ and $0.8L$ and $L$; then from $B$ to $C$ in three parts, using limits 0 and $0.2765L$, $0.2765L$ and $0.7235L$, and $0.7235L$ and $L$; then from $C$ to $D$, where the terms become equal to zero. Integrating as above and solving the resulting equation for $R_1$ it is found that $R_1 = 0.389W$, where $W$ is the load on each span.

The table under Fig. 1 gives the values of all reactions and moments at the dangerous sections as determined by this method of analysis, together with the same quantities as found in the usual way—by the formulas for beams of uniform section.

It might be supposed that in the case of a T-beam the variation in moment of inertia and the resulting error in computed moments would be great owing to the fact that very different sections are opposed to positive and to negative moments. For this reason such a case will be considered.

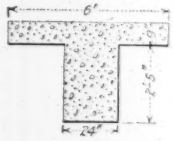



FIG. 2

It will be convenient to assume a girder of definite form, as shown in Fig. 2. The dimensions are taken from a railroad viaduct as actually constructed. It will be assumed that the amount of steel at the supports is just sufficient to give balanced reinforcement. Using values given by the Joint Committee specifications, the required area of steel is found to be 8.4 sq.in. and the resisting moment of the beam at working stresses 4,535,000 in.-lb. Assuming as before that the beam is to be designed for dead-load only, and on the assumption of uniform moment of inertia, we would have as the maximum positive moment provided for in the first span, $\frac{0.08}{0.10} \times 4{,}535{,}000 = 3{,}628{,}000$ in.-lb.; and as the maximum in the middle span, $\frac{0.0316}{0.10} \times 4{,}535{,}000 = 1{,}432{,}000$ in.-lb. Computing by T-beam formulas, we find that the steel areas at these points should be 6.48 sq.in. and 2.49 sq.in. respectively. The area of steel will be taken as constant between points of inflection, as formerly.

The moments of inertia are found from the transformed sections, the steel being replaced by an area of concrete 15 times as great. The concrete of the original section is not assumed to be effective in tension. Fig. 3 shows the beam as considered. The maximum moment of inertia is assumed as unity and the other values expressed as ratios thereof. The method of analysis is

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exactly the same as for the first case, and the results are given in the table under Fig. 3.

In view of the fact that the above analyses assume a somewhat greater variation in moment of inertia than would be likely to occur in practice, it is probably safe to say that the greatest error due to the use of the ordinary theory would not exceed 50% in the case of the positive moments nor 10% in the case of the negative moments. The former error would be on the side of safety, the latter on the side of danger. In a beam of

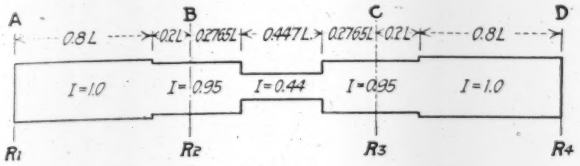



FIG. 3

| Method of Computation | $R_1 = R_4$ | $R_2 = R_3$ | +M in First Span | —M at Supports | +M in Center Span |
|---|---|---|---|---|---|
| Usual method | 0.4000W | 1.10W | 0.08Wl | 0.10Wl | 0.0250Wl |
| Exact method | 0.3965W | 1.1035W | 0.07855Wl | 0.1025Wl | 0.0225Wl |
| % error..... | 0.9 | 0.3 | 1.8 | 2.5 | 11.1 |

the kind here considered the concrete stress in the under side of the beam at the supports is generally the hardest to keep within specified limits, even when a higher compressive stress is allowed there than at other points. Some designers attempt to get around this difficulty by deepening the girder at these points or by providing compression reinforcement, or both. The effect is to increase still further the moment of inertia and hence the bending moments there. As a consequence the concrete stress is always likely to be higher than expected.

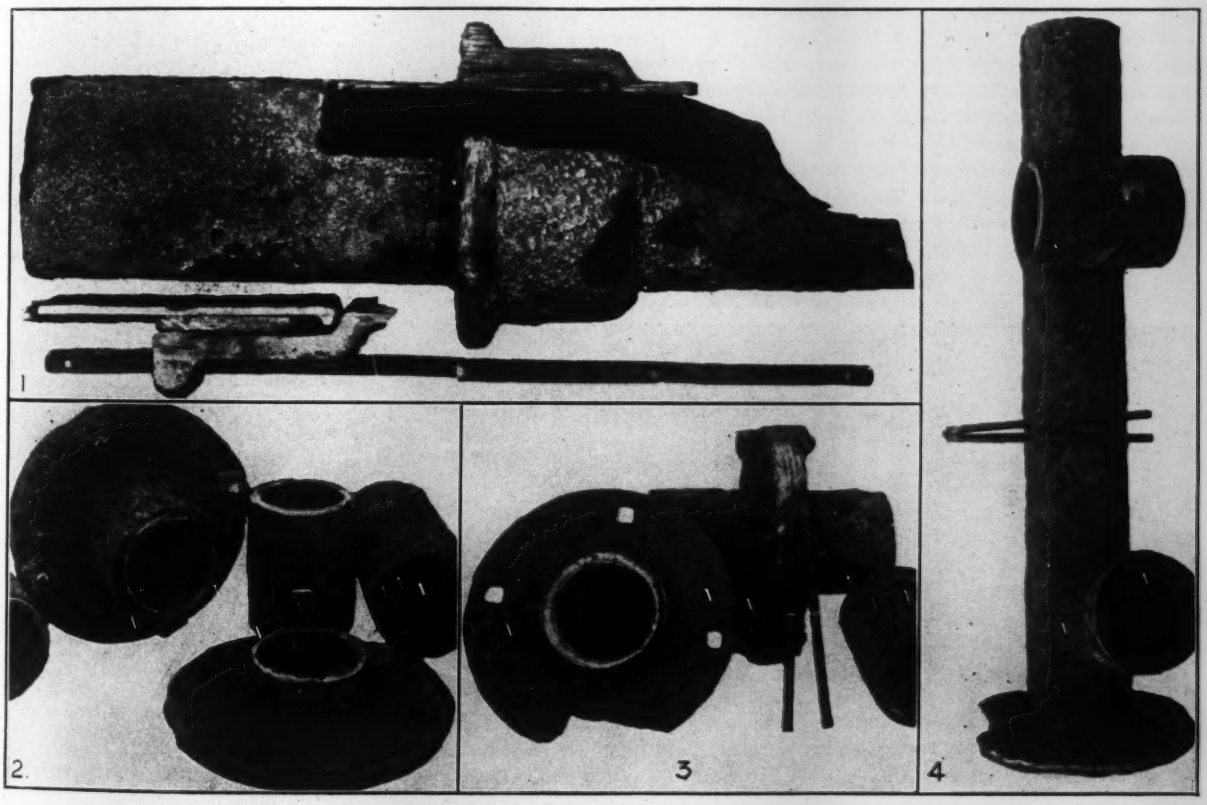
## Relics of Early Water-Works

Philadelphia enjoys the distinction of having the first American municipal water-works to use cast-iron mains to any considerable extent. The earliest of these were laid in Water St. in 1804 as an experiment. The pipe was imported from England, at a time when popular prejudice against the use of iron pipes for potable water-supplies still ran high.

The accompanying illustrations show some of these early water mains, although none date to 1804. Fig. 1 shows some 6-in. bell and spigot jointed pipe laid in 1827 in Locust St., east of 8th St., which was removed in the latter part of 1915. The other illustrations show sections of some older 4½-in. flanged pipe laid in Chestnut St., between Broad and 15th St. in 1817. All this pipe was still in good serviceable condition and was removed only because in one case the main interfered with the construction of a new sewer, and in the other to install a larger pipe previous to laying a new pavement.

An analysis of the incrustations on the 1827 pipe is of interest. It was chiefly oxide of iron and sand, together with some organic matter. The exact chemical analysis was: Iron oxide ($Fe_2O_3$), 77.40; sand (insoluble matter, $SiO_2$), 7.85; moisture, 2.16; volatile matter (organic), 11.95; total, 99.36.

The older pipe was probably imported from England, but the 6-in. bell and spigot pipe very likely came from the foundry of Samuel Richards, established in Philadelphia in 1820. If cast in accordance with the specifications drawn by Frederick Graff in 1816, this pipe was only $\frac{5}{16}$ in. thick as originally laid in the ground.




FIGS. 1 TO 4. WATER-WORKS RELICS DUG UP RECENTLY IN PHILADELPHIA: IRON WATER-MAINS AFTER NEARLY A CENTURY OF SERVICE

Fig. 1—A 6-in. bell and spigot pipe laid in 1827. Figs. 2, 3 and 4—Parts of a 4½-in. main laid in 1817

# Tests of 18,000-Hp. Tallulah Falls Turbines

By E. Lauchli*

Tests of the 18,000-hp. vertical-shaft hydraulic turbines in the Tallulah Falls plant of the Georgia Railway and Power Co. (described in *Engineering News*, Jan. 29, 1914) were completed during the year past, and the results may now be recorded. The maximum efficiency of the three units tested was 88.01% and the average at 80% load was 87.82%.

The specifications required that each unit should develop 12,800 hp. at 514 r.p.m. under 580 ft. head, with an efficiency of not less than 83%. Each unit, see Fig. 1, consists of (1) a 10,000-kw. 6,600-volt alternating-current generator, with oil-pressure bearing and exciter carried on top of the generator, and (2) an 18,000-hp. turbine, with cast-iron scroll case, steel wicket gates, bronze runner and steel-lined draft tube. Oil for the pressure bearing is furnished by two pumps, each capable of carrying the load. One pump is mechanically driven from the main shaft of the unit; the other is driven by an electric motor.

Few hydro-electric plants have been designed so particularly with a view to rendering easy and accurate the recording of turbine-discharge measurements as has the plant under consideration. The specifications called for weir-discharge measurements to be made in connection with the efficiency tests, using the Francis formula corrected for the velocity of approach. The weir, installed in the tailrace of the plant, as shown in Figs. 2 and 4, is 30 ft. long, with end contractions. It was built of heavy timber carefully calked and carried on a concrete weir anchored and sealed into the substrata. Still boxes were provided at each end, connected with a perforated submerged pipe, 8 ft. above the weir, as shown in Fig. 4. Before carrying on the tests, the leakage from different sources was determined by measuring water over a 12-in. weir. The leakage did not exceed ¼ cu.ft. per sec.

The depth of water over the crest was measured with improved Gurley hook gages, located in the still boxes. As it was anticipated that water would pile up on the far side of the weir, readings were taken simultaneously in both still boxes, the actual discharge being computed from the average discharge as recorded in each still box.

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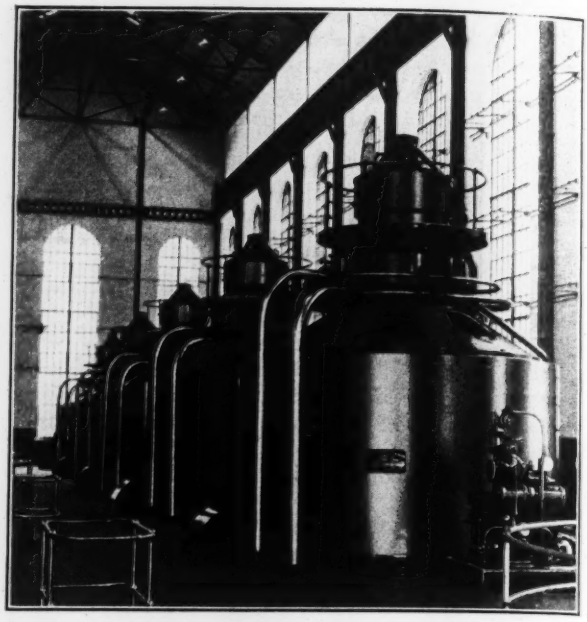


FIG. 1. 10,000-KW. GENERATING UNITS IN THE TALLULAH FALLS STATION

In general, readings of the recording instruments were taken independently by representatives of the power company and of the turbine manufacturers, the S. Morgan Smith Co. In order to make due allowance for the velocity of approach in the Francis formula for weir discharge, a cross-section of the channel of approach in the tailrace was taken 8 ft. above the weir, as shown in Fig. 4, and the additional head computed therefrom. Also, in order to gain an idea as to the distribution of the velocity of water in the channel of approach, velocity readings were taken with a Gurley current meter, 8 ft. above the weir, at various stations, the velocity curves obtained in the center of the weir for discharges of approximately 200, 250 and 300 sec.-ft. being shown in Fig. 4.

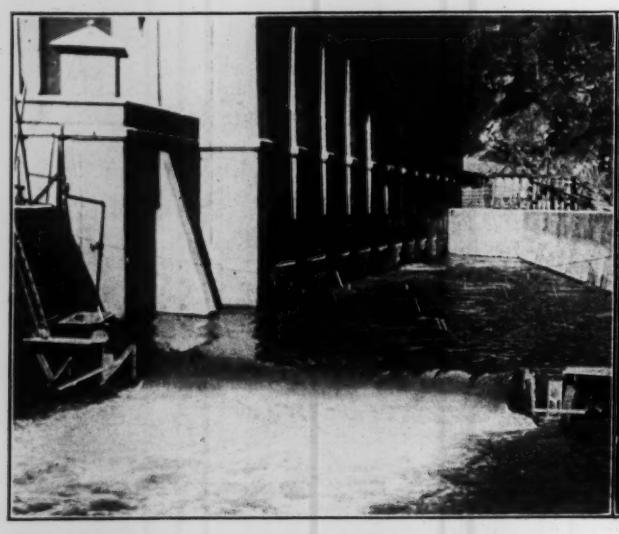


FIG. 2. TAILRACE WEIR AT TALLULAH FALLS

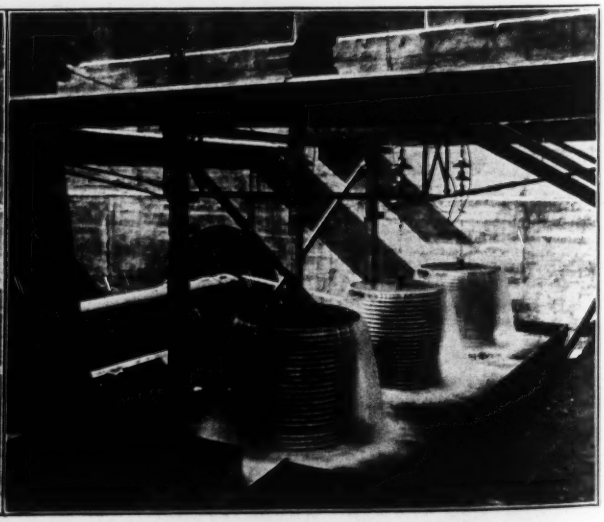


FIG. 3. WATER RHEOSTATS FOR GENERATOR LOAD

The weir measurements were checked with venturi meters, furnished by the Builders Iron Foundry and installed on the penstocks near the power house. Each meter has a large diameter of 60 in. and a 35-in. throat; the recording instruments comprise manometers giving instantaneous readings and a registering instrument making autographic charts. Pitometer measurements were made by the Pitometer Co., the recording instruments being installed at the throat of the venturi meters, where the flow is most perfect.

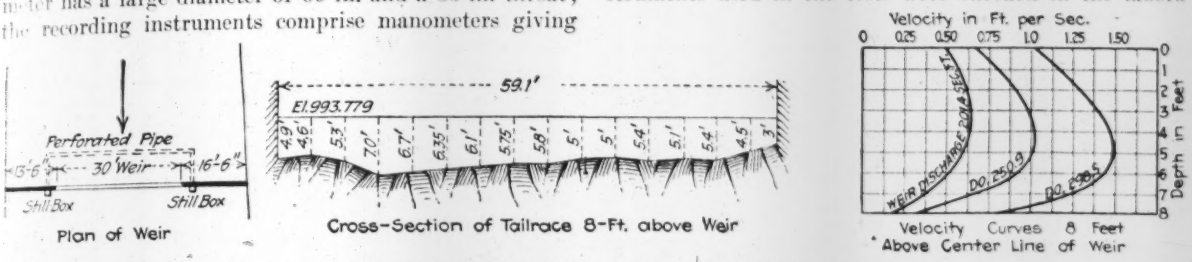



FIG. 4. VELOCITY CURVES ABOVE WEIR, TALLULAH FALLS PLANT

The net head on the unit tested was measured by at least two and usually three calibrated pressure gages located at the entrance of the scroll case. That the weir measurements were carried on successfully and carefully was subsequently proved by comparing the weir discharge with the corresponding discharge recorded with the venturi tubes and pitometers. The readings of the latter meters are not given here, but in general they were about 2 to 3% lower than the weir and venturi-meter readings. From the accompanying table it becomes evident that in general the weir readings are slightly lower than the venturi-meter readings, but the variation is small indeed, and always less than 1%. The venturi-meter readings check closely with discharges as computed by the Bazin formula for weir discharge.

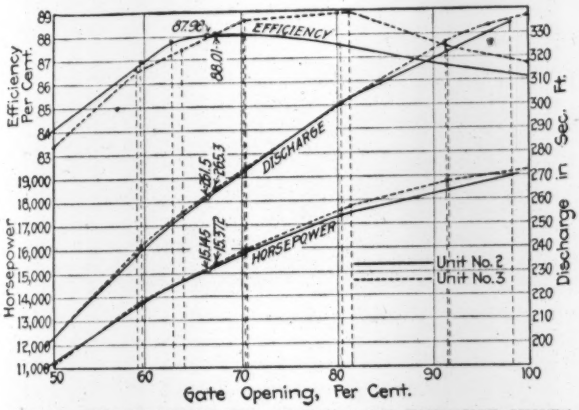



FIG. 5. CHARACTERISTIC CURVES OF TURBINE UNITS 2 AND 3
Referred to 580-ft. head

COMPARISON OF FLOW BY WEIR AND VENTURI METER

| Test No. | Weir Discharge, Sec.-Ft. | Venturi-Meter Discharge, Sec.-Ft. | Difference, Sec.-Ft. |
|---|---|---|---|
| 1 | 201.4 | 201.0 | + 0.4 |
| | 238.1 | 238.7 | —0.6 |
| | 270.7 | 271.2 | —0.5 |
| | 250.2 | 249.6 | + 0.6 |
| 2 | 204.0 | 205.0 | —1.0 |
| | 241.7 | 242.0 | —0.3 |
| | 271.75 | 273.0 | —1.25 |
| | 254.7 | 253.0 | + 1.7 |
| 3 | 206.5 | 208.0 | —1.5 |
| | 244.4 | 245.7 | —1.3 |
| | 273.2 | 275.0 | —1.8 |

The load on the generators was obtained by means of two three-phase star-connected water rheostats, Fig. 3, which gave approximately equal loads on the three phase at approximately unity power factor. Water for the rheostats was taken directly from the penstocks. All instruments used in the tests were checked in the laboratory of the Georgia Railway and Power Co. before and after the tests. During testing, the wiring of the electrical instruments used was run in as careful a manner as possible in order to avoid all stray-field errors. The generator losses on each generator used were obtained from tests by the General Electric Co. at its factory. Readings were taken simultaneously at all points, at one-minute intervals, the starting time being given by flashing all station lights. At least five accurate readings with steady load were taken at 50%, 60%, 70%, 80% and 90% gate opening, also as many readings at 100% gate opening as were necessary. The characteristic curves of two units, as tested, are given in Fig. 5. The maximum efficiency was 88.01%, and the average 87.82% at 80% load, for the three units tested.

## Mississippi Water Terminals

River ports and water terminals were discussed at the second annual meeting of the Mississippi Valley Terminal League, held at St. Louis, Mo., Feb. 16. It appears that of 36 river ports all but two own some part of the waterfront. At 22 of these ports no water terminals are being built, while 14 have them or are building them.

Minneapolis, Minn., has built 1,300 ft. of municipal dock wall and has authorized a bond issue of $75,000. Davenport, Iowa, has spent $177,000 on works estimated to cost $1,000,000 when completed. The amount is raised by 4% municipal bonds, and the plant is made self-supporting by means of rentals and leases. Muscatine, Iowa, is spending $35,000. Hamilton, Ill., has a dock that was built for it by the Mississippi River Power Co. at a cost of $6,000. Kansas City, Mo., has built a municipal dock and authorized a bond issue of $75,000. Baton Rouge, La., has accepted plans for the first unit of proposed terminals, to cost $75,000. New Orleans, La., is spending $2,000,000 raised by bonds.

St. Paul, Minn., has plans under consideration for work estimated to cost $50,000. At Des Moines, Iowa, some work is being done. At Keokuk, Iowa, the preliminary filling is being done on the waterfront. Peoria and Quincy, Ill., propose bond issues of $100,000 and $75,000 respectively. St. Louis, Mo., proposes two municipal docks with railway connections, to be provided for by a bond issue of $2,000,000. At East St. Louis, Ill., about $75,000 is being expended, the amount being raised by private enterprise.

More detailed information regarding many of the projects was given in *Engineering News*, June 3, 1915.

# Concrete Highway Trestle

*SYNOPSIS—The longest concrete highway trestle ever built is probably that which the California State Highway Commission has nearly completed near Sacramento, over 3 mi. in length. The trestle has been constructed on the unit system. It is built with precast concrete piles, and the floor is made up of four separate precast slabs placed side by side to span the space from bent to bent. The methods of piledriving, transportation and erection used were unique. This substantial concrete structure, 20 ft. in height with a roadway 21 ft. in clear width, has cost about $130,000 per mile.*

The Sacramento River, a stream of moderate size during California's long dry season, becomes during the spring floods, when the melting snows in the Sierras pass down the valley, one of the largest rivers of the United States. A flood flow of over 800,000 cu.ft. per sec. has been recorded. This volume of water is several times as great as the regular channel of the Sacramento will carry. The project for the reclamation and flood protection of the Sacramento Valley therefore provides for leaving a broad channel down the valley unobstructed by dikes. Into this channel, known as the Yolo bypass, the surplus water from the main channel will be discharged in time of flood.

The California State Highway Commission in constructing a state highway across the Sacramento Valley had to plan a road which should not be interrupted at periods of flood, which often last for several weeks. It was necessary therefore that the roadway across the Yolo bypass should be built above flood level and that it should not obstruct the flow of water down the channel.

It was decided that the best and most economical structure to carry the roadway across this channel was a concrete trestle. At one end, however, the final limits of the bypass channel are not positively determined, and at that end it was decided to build a wooden structure on creosoted piles, which would cost less and be more readily removed in case of necessity than a concrete structure.

FIG. 1. VIEW OF COMPLETED TRESTLE ACROSS YOLO BYPASS

The ground crossed by the trestle is a swamp of uniform character, so that it was possible to drive piles accurately and space the bents uniformly. It was decided that the concrete trestle should be built by driving precast concrete piles, four piles to a bent, molding a concrete cap on the heads of each bent and then placing precast floor slabs on these bents, leaving the parapet of the structure and the wearing surface for traffic as the final steps in construction. The work of constructing the trestle is described in detail as follows from information given by B. C. Gerwick, the resident engineer in charge of the work.

### Layout of the Concrete Casting Yard

As may be seen by the accompanying sketch map, the west end of the trestle intersects the main line of the Southern Pacific. A spur track was built here, and bunkers were erected with a capacity of two carloads of sand and four carloads of rock. The railway delivered sand and rock in bottom-dump cars which discharged into a hopper under the track, and a bucket elevator hoisted the material into the bunkers. A yard was laid out for casting the piles and floor slabs. The concrete piles, 32 to 50 ft. long, weighed from 3 to nearly 5 tons, according to their length; and the floor slabs weighed 5¼ tons for the inside slabs and 6¼ tons for the outside slabs.

Corrugated steel was used for reinforcement and was all assembled in units before placing. This work was very accurately done, and the cost was comparatively low. The average cost of bending, assembling, and placing in the forms was about $7.50 per ton, and some work was done as low as $6.80 per ton.

### Methods Used in Piledriving

Each bent of the concrete trestle has four piles, the two inner piles vertical and the outer piles driven on a batter of 1:8. The concrete piles used are 14 in. square with chamfered corners. There is a reinforcing rod at each corner and spiral hooping. The lower 5 ft. of the pile is tapered. Prior to the driving of the piles, numerous test borings and test piles were put down along the line of the trestle. An attempt was made to have all the concrete piles of such a length that they could

be driven to the desired grade and afford a resistance to permit a safe loading of 20 tons each.

Piles of five different lengths were cast, as follows: 32 ft. long, 860; 35 ft. long, 1,084; 50 ft. long, 524; 45 ft. long, 236; 50 ft. long, 220. By using these piles of varying lengths it was possible in nearly all cases to drive a pile to the final grade desired and obtain the required resistance. The soil encountered was a mixture of sand and clay, with some adobe on top, and varied considerably in bearing power. The penetration under the final blow in some cases was as much as ½ in.; but in others the last 20 or 30 blows lowered the pile only an inch, and in a very few cases it was necessary to cut off the piles.

The contract for the work was let July 31, 1914, and a month later the machinery was being set up. The first equipment arrived at the casting yard on Sept. 5. Pile-driving began on the creosoted wood-pile trestle at the west end on Nov. 1, and the first concrete pile was driven on Jan. 3, 1915. The last concrete pile was driven Oct. 1, 1915.

### Methods of Handling the Piledriver

The piledriver used was 60 ft. high with swinging leads. When the work began, the swamp to be crossed by the trestle was dry. This swamp on the line of the trestle is almost a dead level, the elevation varying only from +12.5 to +8.6 in the 3¼-mi. length of the trestle. The desired elevation of the top of the concrete piles was 28.9, so the trestle floor is 16 to 20 ft. above the ground. To carry the piledriver over the soft swamp, 12x12-in. by 24-ft. skids were laid directly on the ground with 2x6-in. greased ways on these for the piledriver to slide on. The driver was backed away from the trestle as it worked, and the skids were pulled ahead with a team.

In February the swamp was flooded and a different method of working became necessary. A temporary wooden trestle was then driven ahead of the concrete trestle to carry the piledriver. This temporary trestle had bents placed 9½ ft. apart, four piles to each bent, and the piles were driven 8 to 12 ft. into the ground and projected about 8 ft. above the ground, which was enough to keep the caps clear of the water. A small piledriver drove the piles for this temporary trestle and placed the caps and stringers. The large piledriver, when it moved back from driving a bent of concrete piles, pulled the piles of the two temporary bents which it had passed over, and the piles, caps and stringers were floated ahead to the small driver.

### Cushioning the Hammer Blow on the Pile

There were four ⅞-in. reinforcing rods in each concrete pile, and they were left projecting 20 in. above the head when the pile was cast, to make a proper bond with the concrete caps, which were later cast in place. These projecting rods necessitated special arrangements in driving the piles to avoid bending the rods. Immediately on top of the piles was placed a 4-in. fir block, or in case of the long piles two blocks, through which holes had been bored for the projecting rods. On top of this wooden block was placed a structural-steel head with a sliding cast block. Later a cast-steel head was used, holes being cored through it to admit the rods. On top of this cast-steel follower was placed a 14x14-in. block

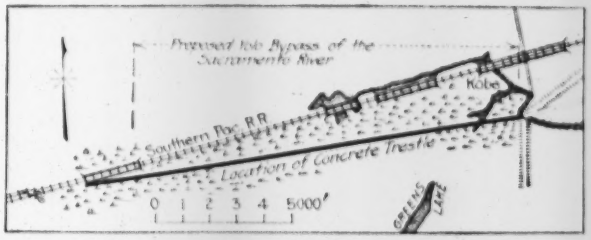



FIG. 2. SKETCH MAP, SHOWING LOCATION OF CALIFORNIA STATE HIGHWAY ACROSS YOLO BYPASS

of Australian ironwood, to receive the blow of the hammer. This block, chamfered at the upper end and fitted with a cast-steel cap, shattered rapidly under the shock of the hammer, lasting from one to five days, or a maximum of about 100 piles driven. It was found that when the fiber of the block broke down, much of the force of the hammer's blow was lost.

The concrete piles withstood the shock of driving in splendid shape, very few of the pile heads breaking down. When any of them did break, it was generally easy to find the cause, such as the head being out of square, or the cushion block too hard.

A drop hammer was used for driving the creosoted piles and was tried on the concrete piles, but the piles shattered badly under it. A No. 1 Vulcan hammer was then substituted and worked well. This ram weighed 5,000 lb., had a drop of 3½ ft. and struck 56 blows per minute.

As the concrete-slab floor units were cast of uniform length, it was necessary to drive the piles very accurately. The piles were lined up with transits, one on the center line of the bridge and one on a parallel base line. When

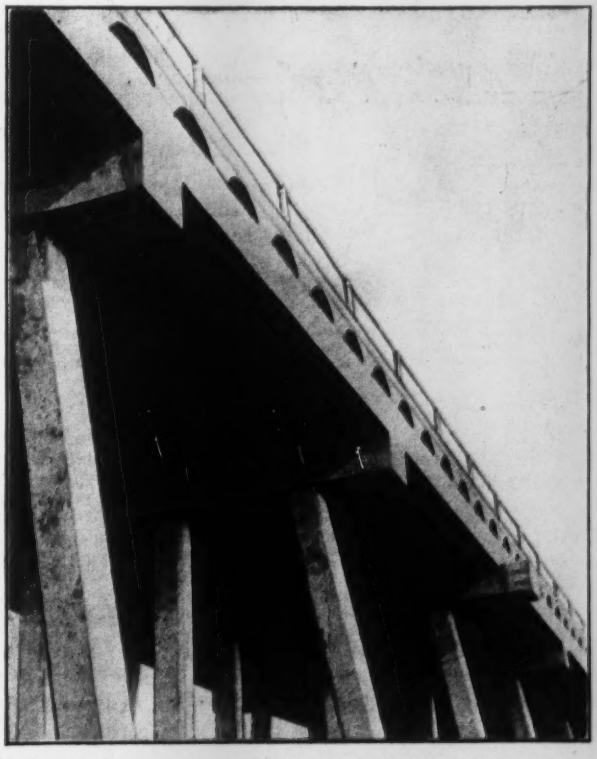


FIG. 3. VIEW OF UNDER SIDE OF YOLO BYPASS TRESTLE, SHOWING FLOOR CONSTRUCTION

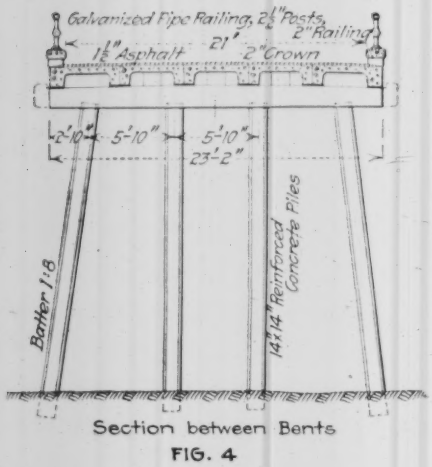
the ground became flooded so that the transit on the base line could not be used, direct measurement was relied upon to obtain the distance to the head of the pile. This spotting of the piles was done very rapidly, as may be seen by the fact that 30 to 32 piles of 35-ft. length were usually driven in 8 hr. The driver had also to pull the temporary trestle. The alignment of the piles is remarkably good. The maximum allowable variation of the head of the pile in the cap was 2 in. Only a few piles had to be sprung slightly to come within this variation.

As every engineer knows who has had to do work involving the use of long precast reinforced-concrete piles, it is essential that they be carefully handled to avoid excessive bending stress. The piles after being thoroughly seasoned in the yard were hauled out at night to the end of the completed trestle. In order to handle these handled directly to the driver. All the longer piles were lifted with two lines, one being attached well back from the head of the pile and a second at the rear quarter-point.

### Temporary Track for Handling Material

As soon as a bent of four concrete piles was driven, a temporary deck was placed on the two south piles. The deck consisted of a 10x12-in. cap bored to admit the rods projecting from the heads of the piles and on this cap rested stringers spanning the 20-ft. space from bent to bent, while on these stringers were laid crossties and 18-lb. rails for a 3-ft. gage track. On this railway ran the cars which brought the piles and other construction material to the end of the work.

The next operation was the casting of the reinforced-concrete cap on the north half of the pile bent. Wooden




Section between Bents

FIG. 4

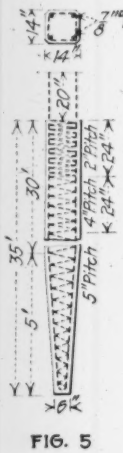



FIG. 5

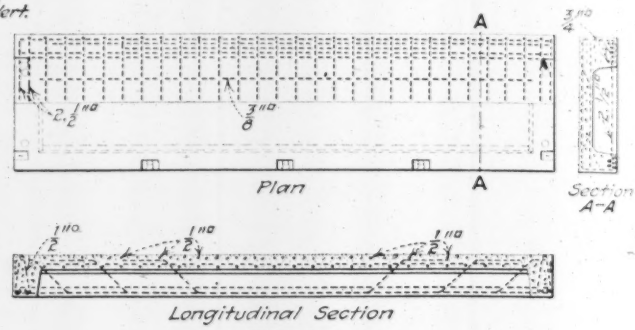



FIG. 6

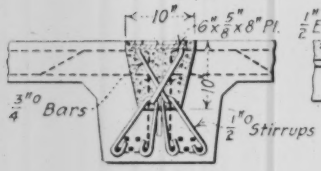



FIG 7

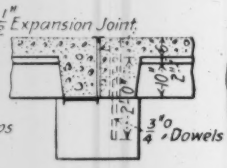



FIG 8

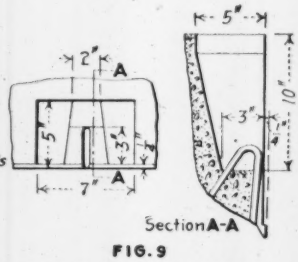



FIG. 9

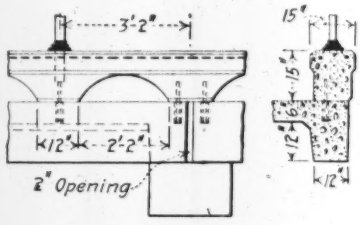



FIG. 10

FIGS. 4 TO 10. VARIOUS DETAILS OF TRESTLE CONSTRUCTION, YOLO BYPASS, CALIFORNIA

Fig. 4—Cross-section. Fig. 5—Reinforcement of 35-ft. pile. Fig. 6—Plan and sections of outside floor slab. Fig. 7—Slab connections at junctions over trestle caps. Fig. 8—Details of expansion joints. Fig. 9—Loop for lifting and anchoring floor slabs. Fig. 10—Longitudinal elevation and section of wheel guard

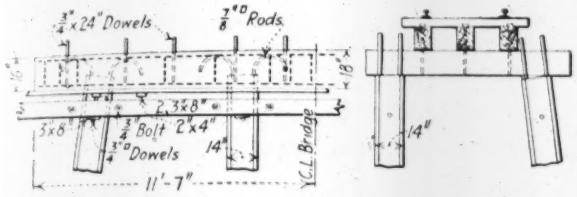
long piles, the piledriver was equipped with a 12x12-in. boom 48 ft. long, the inner end of which was set in a foot block between the standing leads of the driver. The swinging leads were so arranged that they could be swung to one side out of the way. When the boom was to be used for handling piles, the hammer line was used as a boom line and the pile line as a fall line, while a line to a niggerhead was used to swing the boom.

At first the piles were hauled from the yard to the end of the trestle at night and lowered by the boom on two horses placed beside the trestle, so that the piles were parallel with the bridge, the piles resting on the horses at about their quarter-points. Later, when the swamp was flooded, this plan was changed, and the piles were hauled out to the end of the trestle in the daytime and forms were built around the heads of the piles, which were allowed to project 6 in. into the cap. The rods projecting from the top of the piles were bent over into approximately semicircular form. The other reinforcing rods were placed in the form and left projecting toward the south end. The half-cap was then concreted. This work was done at night, the mixer and concrete material being carried on the temporary track on the south half of the trestle, which had to be kept open for delivering piles to the driver during the day.

These caps were allowed to season for 30 days, and then the two floor slabs for the north half of the floor were brought from the yards, set in place on the caps and grouted. The temporary track was then removed from the south half of the trestle, and the ties and rails were

relaid on the floor slabs of the north half. It was then possible to cast the south half of the trestle caps. After these had seasoned for 30 days, the floor slabs were brought forward and placed upon them.

As may be seen from the accompanying drawings, each floor slab has the general form of an inverted box. The




Caps are first poured on North Half of Bridge
All Work from Deck of Bridge Concrete mixed in Mixer on Car on Track, deposited in Form by Means of Chute Concrete placed Nights when Track was clear

FIG. 11. CROSS-SECTION, SHOWING CONSTRUCTION OF NORTH HALF OF CAPS OF PILE BENT

floor is 6 in. in thickness, with projecting flanges of a total depth of 18 in. Great pains was taken in the design of the slabs and caps, so that the completed trestle is a monolithic structure, although the separate parts are assembled as precast units. Rods were left projecting from the top of the trestle caps, and 3-in. holes were cored in the floor slabs, so that as the floor slabs were set they dropped over these rods. The holes were later filled with grout. For convenient handling of the floor slabs, four round-iron links were cast in each and were set in slots at the ends of the slabs. After the slabs were set in place these links were connected by hook bolts and a plate, and then the whole slot was filled with grout.

The sides of the slabs were made flush at the bottom for a depth of 4 in. and recessed ¼ in. above. When the slabs were set in place, a half-inch opening closed at the bottom was thus left between adjacent slabs, and this was later filled with grout.

At every third bent of the trestle was placed an expansion joint. Here of course the connecting hook bolts referred to were omitted. A half-inch opening was left in setting the slabs, and in this opening was inserted a fold of No. 16 galvanized iron. The expansion slabs, like all the other floor slabs, were bedded in cement mortar, which was applied to the trestle caps before the floor slabs were set. Movement takes place, however, between the mortar and these slabs at the expansion joints. The bottom of the slab is very smooth and somewhat oily from the forms, so that it slides readily on the mortar layer on top of the trestle cap. The movement in this expansion joint is noticeable under even slight changes in temperature. The highest temperature since the slabs have been set is 113° and the lowest 28°.

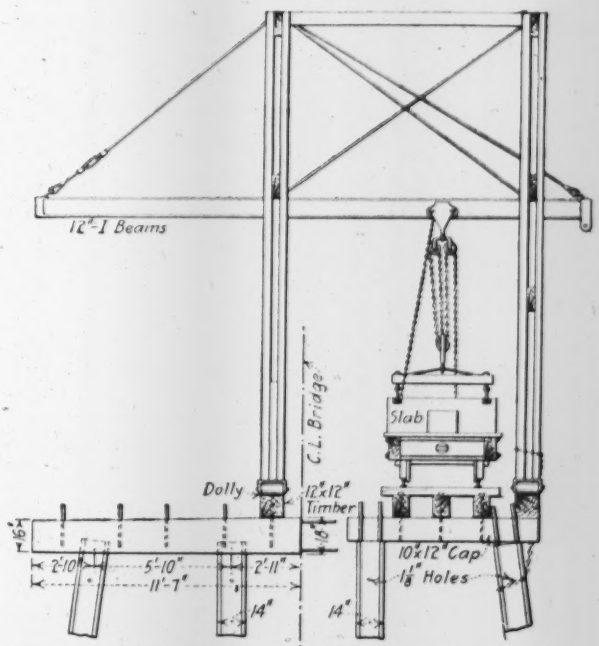
### Equipment for Transportation of Material

For hauling the piles, slabs and other material from the casting yard to the end of the trestle, special cars were used, consisting of two sets of car trucks connected with a reach, and two 12x12-in. timbers set on top of the trucks. Two floor slabs made a load for one of these cars, and a train of three loaded cars was pulled by a gasoline locomotive. At the outset of the work one 5-ton gasoline driven locomotive was used; and as the distance of haulage increased, a similar 7½-ton engine was added. These gasoline locomotives worked three 8-hr. shifts and gave excellent satisfaction. While the grade on the concrete trestle was level, a temporary construction trestle extended from the yard on a 0.4% grade which the train had to ascend.

### Parapets and Roadway Surfacing

For setting the slabs on the trestle cap a gantry crane traveler was used carrying two 5-ton chain blocks. When used to set the first row of slabs on the north side of the trestle, the traveler ran on 12x12-in. stringers which on one side rested on the inner end of the concrete cap and on the other side rested on the outer side of the cap on the temporary construction track, as shown in Fig. 12. To prevent the traveler, from tipping, it was chained down when setting the first slab. After this slab was placed, however, a plank was laid on top of it, and then a traveler was used spanning the entire trestle. With these travelers it was possible to set the floor slabs very carefully and accurately. Accuracy in setting was essential, as the bottom edges of the longitudinal joints had to be flush in order to grout these joints. Accuracy in placing the slabs was essential also, on account of the method of fastening used.

In casting the outside slabs for the floor a row of 1½-in. holes 6 in. deep was cored in the slabs. After the slabs had been set, rods 12 in. long were inserted in these




Gantry Traveler straddles Track and overhangs North Side. Floor Slabs are run underneath Traveler, picked up by two, five-ton Blocks, and moved over into Place; then lowered on Cap over Dowels cast in Concrete. When two North Slabs are set and grouted for some Distance, say 100 Bents, Track is then thrown over on Slabs. Temporary Caps and Stringers are taken ahead for further Use. The Forms for South Side Half Caps are then placed and concreted in same Way as the North Side

FIG. 12. TRAVELER FOR SETTING FLOOR SLABS

holes and grouted in place. These projecting dowels hold the parapet in place on the floor. The parapet is of simple form, 15 in. in height with arch openings. Besides acting as a curb, it supports a galvanized-pipe railing, the top of which is 43 in. above the floor. The wearing surface of the roadway is a layer of asphalt

1½ in. thick. The roadway has a 2-in. crown at the center, this crown being formed by casting the trestle caps 2 in. higher at the center than at the sides.

The unit system of construction adopted in building the trestle not only produced a permanent structure built of uniformly high-quality material at a low cost, but enabled the work to be prosecuted very rapidly. The first concrete pile for the trestle was driven on Jan. 3, 1915, and the last concrete pile on Oct. 1. As may be gathered from the description, the different parts of the work were being carried forward at the same time the piles were being driven, so that the entire floor of the trestle with wheel guard in place was complete on Dec. 11, 1915; and on Dec. 22 the first automobile was driven across the bridge. There still remains to be constructed a bascule bridge of short span crossing a navigation channel at the east end of the trestle. A temporary roadway has been built around this, however, and the whole structure is expected to be open for traffic in February, 1916.

In carrying on its work of highway construction the California Highway Commission furnishes to contractors the crushed rock, sand and cement for concrete, and any steel used for reinforcing. A special freight rate of about one-half the regular rate is made by the railways of California for material for use on the state highways. This is one reason why the commission has adopted the policy of furnishing the materials to the contractors. In the Yolo bypass trestle the total amount of material is 21,692 tons of crushed stone, 12,553 tons of sand, 32,000 barrels of cement and 2,200 tons of reinforcing steel. The cost of these materials was in the neighborhood of $148,000. The payments to the contractor for the trestle, the Graff Construction Co., will aggregate about $246,000. The total cost of the trestle will be under $395,000. The total length of the trestle is 16,310 ft., or 3.09 mi. One-seventh of this distance is of wooden construction. The cost of this, however, is included in the contract price quoted, so that the total cost of the concrete trestle is in the neighborhood of $130,000 per mi. Austin B. Fletcher is the Chief Engineer of the California Highway Commission, and William S. Caruthers is division engineer of the division in which the Yolo bypass trestle is included.

## Property Owners Asked To Slope Street-Railway Cut

Kansas City, Mo., has spent five years and about $400,000 digging a cut for a 60-ft. street carrying two tracks of the Metropolitan Street Ry. Co. The cut is 50 ft. deep, three city blocks in length, in rock and earth. The line was designed to be an outlet for traffic from the new Union Station to the south residence district. About four months ago, after the work had been completed, a large rock broke off and rolled down the steep slope, breaking a trolley pole at the bottom. Six weeks later another rock weighing several tons broke loose and struck the track with sufficient impact to bend the rails. The board of control of the railway thereupon refused to operate cars through the cut until the latter was made safe.

In excavating the cut the contractors used heavy charges in blasting, which shattered the rock badly. The sides were left almost vertical. The formation is made up of strata of limestone with intervening strata of shale and dirt. When exposed to the weather, the shale disintegrates so that the rock ledge above is subjected to cantilever stresses, causing it to fail. It is stated that

STRATIFIED ROCK IN CUT FOR STREET RAILWAY AND STREET IN KANSAS CITY

the sides of the cut should be graded to a slope of 1 : 1. The character of the rock is shown in the view.

The railway referred the matter to the city, under the jurisdiction of which the work was done. The city engineer, having no jurisdiction outside the slope lines, referred the matter to the legal department of the city and was advised that there was an ordinance on file which called upon the adjacent property owners to grade their property to such a slope as would make the street safe.

The park board in grading back for the station plaza will grade the street for about half its length on the east side of the cut. Only one property owner on the west side has announced his intention of grading it back. In the meantime the city has been unable to lay the asphalt pavement. Probably the matter will be held in abeyance until the park board has completed its grading, when some action may be taken to force the abutting property owners to observe the ordinance.

# High Sulphur Does Not Injure Openhearth Steel

By J. S. Unger*

*SYNOPSIS—As a result of the unwillingness of the Society of Automobile Engineers to accept high-sulphur steel specifications, the Carnegie Steel Co. has carried on a long series of practical studies of the behavior of steels alike except for sulphur content. Some of the results were presented in a paper before the annual meeting of the society in New York, Jan. 6, 1916, and are reviewed here. Rivets, chains, tubes, channels, plates, sheets, rails, axles, drop forgings, etc., were made, and the higher-sulphur pieces were generally as good as those of lower content.*

The manufacture of steel in quantities of several tons at one time began when the bessemer process was introduced about 50 years ago. Chemical analyses at that time were crude, or not made at all. Failures due to poor raw material or improper metallurgical treatment were common. Sulphur was largely blamed for such results, and a strong prejudice against it was established. Rarely has it been questioned, nor have many efforts been made to establish the truth, until at the present time few are ready to believe that sulphur up to a reasonable amount, say under 0.100% does not affect, or at the most only slightly influences, the working properties during manufacture or the quality of the finished steel.

The purpose of this investigation was to prepare steels of different degrees of hardness, each containing varying amounts of sulphur, then work these steels into finished products and examine them carefully during the manufacture and after completion for difference in quality.

Three heats of steel of 68 tons each were made. These were low-sulphur basic openhearth steels of soft, medium and moderately hard varieties of approximately 50,000, 70,000 and 90,000 lb. tensile strength, the carbon contents being 0.09, 0.32 and 0.51% respectively. No selection of stock or furnace was made, the furnaces being taken at random. The heats were cast into twenty-four 18x20-in. ingots, of 6,300 lb. each. Twelve ingots from each heat were used in the investigation, or a total of 100 tons. After a discard had been made to eliminate any highly segregated or streaked condition in the steel and the regular waste provided for, about 50 tons of steel was used in the tests carried out.

The sulphur content of the ingots was increased progressively by adding amounts of sulphur to different ingots from the same heat, raising the amounts in the higher-sulphur ingots of the series greatly beyond that ordinarily found in commercial steels. The sulphur additions were made in the pure powdered form to the ingots during pouring. The aim was to obtain steels that, excepting sulphur, would be alike in manufacture and composition, thus keeping out any variables and furnishing an opportunity to study the effect of sulphur alone.

The ingots were rolled into such sizes as would be needed to fabricate the different steels, either by hot or cold working, into such finished articles as they were best adapted to by composition. Sheets, wire products, rivets, chains, tubes, channels, plates, rails, axles and drop forgings were made. Owing to the great variety of materials made, it was not always possible to use steel of the exact composition ordinarily employed for a certain purpose, as the steel was sometimes a little harder or softer than required. This feature does not, however, influence the comparative results. The manganese of the three heats was 0.43, 0.62 and 0.67% respectively.

The ingots were heated to 1,250° C. and then rolled in the regular way, no attempt being made to give the higher sulphurs any preference. All ingots were rolled, except the two highest-sulphur ingots of the 0.09% carbon heat, containing 0.250 and 0.254% sulphur. These ingots cracked badly in rolling and were removed from the roll tables and scrapped. In rerolling from the blooms, billets or slabs into finished material, no trouble was experienced nor any difficulties met.

Table 1 shows the chemical composition of each heat and the sulphur content of each ingot.

TABLE 1. CHEMICAL ANALYSES AND PURPOSES OF HIGH-SULPHUR STEEL

Basic Openhearth Heat No. 81,160:
Chemical analysis—carbon, 0.09; manganese, 0.43; phosphorus, 0.012; sulphur, 0.031.
Sulphur content in ingots—0.030, 0.031, 0.050, 0,060, 0.090, 0.116, 0.140, 0.160, 0.180, 0.250, 0.254.
Purposes for which used—rivets, chains, sheets, wire, tubes and pipe.

Basic Openhearth Heat No. 71,163:
Chemical analysis—carbon, 0.32; manganese, 0.62; phosphorus, 0.014; sulphur, 0.032.
Sulphur content in ingots, 0.032, 0.068, 0.108, 0.146, 0.190, 0.230.
Purposes for which used—drop forgings, channels, plates and tubes.

Basic Openhearth Heat No. 76,185:
Chemical analysis—carbon, 0.51; manganese, 0.67; phosphorus, 0.015; sulphur, 0.025.
Sulphur content in ingots—0.025, 0.055, 0.095, 0.135, 0.167, 0.230.
Purposes for which used—axles, drop forgings, rails and wire rope.

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Some 8-in. 11½-lb. channels were rolled, with sulphur running from 0.032 to 0.230%. No tearing or red shortness was noticeable on the thin flanges of the higher sulphurs.

The ability to weld the soft steel of 0.09% carbon was shown by ¾-in. hand- and machine-made chain. Three pieces of each sulphur content were made. The short pieces of three links each were tested to destruction (see Table 2). The greater average strength of the hand-made chain over the machine-made is not due to better workmanship, but to the fact that the lap in the hand-made chain is almost twice as long.

No trouble was experienced in either the hand- or machine-made chain until 0.110% sulphur was reached, 0.180% sulphur acting similarly. It was found that both could be welded perfectly at the regular temperature and did give good results, but that lowering the welding temperature about 100° C. prevented any cracking or crumbling of the steel in welding.

The evidence seems to show that good and bad welds are more a question of heating and workmanship than of either high or low sulphur, as either kind of weld is found in any of the chains tested.

TABLE 2. BREAKING LOAD OF ¾-IN. CHAIN*

| Carbon Content, % | Sulphur Content, % | Breaking Load, Hand-Made Chain, Lb. | Breaking Load, Machine-Made Chain, Lb. |
|---|---|---|---|
| 0.09 | 0.030 | 31,480 | 32,800 |
| | | 39,630 | 33,300 |
| 0.09 | 0.060 | 40,710 | 35,630 |
| | | 41,470 | 31,600 |
| 0.09 | 0.090 | 40,820 | 23,300 |
| | | 30,940 | 24,000 |
| 0.09 | 0.140 | 40,710 | 26,450 |
| | | 37,690 | 22,700 |
| 0.09 | 0.180 | 32,990 | 31,650 |
| | | 40,390 | 31,200 |

* Rolled $\frac{1}{8}$ in. light in gage.

A number of rivets, ¾ in. in diameter by 2 in. long under the head, of the 0.09% carbon steel were made and subjected to the various hot and cold tests without any apparent differences. Ten bars, having two $\frac{13}{16}$-in holes drilled 2 in. from one end, were riveted together by machine at an estimated temperature of 1,200° C. These bars were then tested with the results shown in Table 3. Practically no difference was observed in the results.

A number of bars from both the 0.32% and the 0.51% carbon heats of various sulphur contents, 2x2 in. by 16 ft. long, were rolled and sent to two prominent automobile manufacturers with a request to drop forge them in accordance with their regular practice into such shapes as would indicate if any difficulties would be encountered in the heating or forging. The flash or excess metal forced out between the dies was not always sheared off, it being allowed to adhere to the forging to show any tearing at the thin outside edges. The appearance of the flash seems to furnish the strongest evidence of the ability of the high-sulphur steels to stand severe hot work.

TABLE 3. SHEARING STRENGTH OF RIVETED JOINTS

| Carbon Content % | Sulphur Content % | Shearing Strength, in Lb. |
|---|---|---|
| 0.09 | 0.030 | 48,900 |
| 0.09 | 0.060 | 48,300 |
| 0.09 | 0.090 | 48,800 |
| 0.09 | 0.140 | 46,700 |
| 0.09 | 0.180 | 47,400 |

When finished these forgings were cleaned by dilute acid or the sand blast and then carefully inspected by the manufacturers and the author to detect any surface or other defects, but no differences could be observed in the high- and low-sulphur steels of either carbon.

A number of No. 26 gage sheets were rolled, the galvanized and afterward made up into spouting. In seaming and beading the spouting, no cracks developed in any specimen, while the coating adhered as closely to one sheet as to another.

A number of milk-can bases were made of 0.09% carbon steel of different sulphur contents. These bases were intended for 5-gal. cans, were 11 in. in diameter and of No. 16 gage sheet. The work done was very severe, being a reverse draw with two operations in the press. No differences were noticed in stamping any of the specimens.

Tensile specimens were machined from the drop forgings of 0.32% carbon, after they had been subjected to the heat-treatment given in Table 4. Similar tests after

TABLE 4. TENSILE TESTS OF HEAT-TREATED KNUCKLES*

| Carbon Content, % | Sulphur Content, % | Elastic Limit, Lb. Per Sq.In. | Tensile Strength, Lb. per Sq.In. | Elongation in 2 In., % | Reduction of Area, % |
|---|---|---|---|---|---|
| 0.32 | 0.032 | 62,250 | 83,250 | 30.0 | 60.0 |
| 0.32 | 0.068 | 49,375 | 78,375 | 31.5 | 62.8 |
| 0.32 | 0.108 | 52,750 | 77,750 | 30.0 | 58.0 |
| 0.32 | 0.146 | 58,875 | 77,875 | 27.0 | 61.3 |
| 0.32 | 0.190 | 51,875 | 76,875 | 28.0 | 53.8 |
| 0.32 | 0.230 | 57,250 | 76,250 | 29.0 | 53.8 |

*Annealed at 900° C., reheated at 829° C., quenched in water and drawn at 538° C.

treatment were made from the 1¼-in. diameter rounds used in drop forging from the 0.32 and 0.51% carbon steels. The results obtained are given in Table 5.

## Casehardening; Machining; Tensile Tests

Short pieces of ¾-in. rounds from the 0.09% carbon steel were packed in ordinary casehardening compound in the same container, then heated to 900° C. for 12 hr., afterward reheated to 860° C. and quenched in cold water.

TABLE 5. TENSILE TESTS OF HEAT-TREATED 1¼-IN. ROUNDS FORGED FROM 2x2-IN. BILLETS

| Carbon Content, % | Sulphur Content, % | Elastic Limit Lb. per Sq.In. | Tensile Strength Lb. per Sq.In. | Elongation in 2 In., % | Reduction of Area, % |
|---|---|---|---|---|---|
| 0.32* | 0.032 | 48,650 | 80,250 | 30.5 | 70.1 |
| 0.32* | 0.068 | 48,550 | 75,550 | 32.8 | 68.8 |
| 0.32* | 0.108 | 46,400 | 75,800 | 30.2 | 68.7 |
| 0.32* | 0.146 | 46,700 | 73,350 | 31.5 | 67.3 |
| 0.32* | 0.190 | 45,450 | 71,550 | 33.2 | 66.3 |
| 0.32* | 0.230 | 45,850 | 70,100 | 31.5 | 65.0 |
| 0.51† | 0.025 | 70,400 | 111,900 | 20.3 | 56.8 |
| 0.51† | 0.055 | 76,300 | 120,800 | 19.7 | 51.3 |
| 0.51† | 0.095 | 73,950 | 119,400 | 19.5 | 51.5 |
| 0.51† | 0.135 | 76,800 | 120,600 | 18.3 | 49.2 |
| 0.51† | 0.167 | 73,200 | 111,750 | 17.5 | 45.4 |
| 0.51† | 0.230 | 66,350 | 106,550 | 20.5 | 44.7 |

*Heated to 830° C., held for 20 min., quenched in water drawn at 600° C. for 30 min.

†Heated to 816° C., held for 20 min., quenched in water drawn at 565° C. for 30 min.

The surface of the bars was roughly polished and the scleroscope hardness determined at several points on the surface. The depth of case and hardness were the same for each bar, regardless of the sulphur content.

Where any machining was done on the finished material, or in preparing test specimens before and after treatment, no material differences were found. The only noticeable thing was that the higher-sulphur steels of any

TABLE 6. THREADING TESTS OF 1¼-IN. PIPE (0.09 CARBON STEEL)

| Carbon Content, % | Sulphur Content, % | Cutting Force, in Lb.* |
|---|---|---|
| 0.09 | 0.031 | 73.5 |
| 0.09 | 0.050 | 70.6 |
| 0.09 | 0.090 | 68.5 |
| 0.09 | 0.116 | 62.3 |
| 0.09 | 0.160 | 61.2 |

* A 22½-in. arm attached to the die ran to a dynamometer.

carbon gave a smoother machined surface than the lower-sulphur under the same condition of feed, speed or depth of cut. Table 6 shows the average force required while threading ten samples of each sulphur content.

Tensile tests of ¾-in. rounds of 0.09% carbon, of 8-in. channels of 0.32% carbon and of axles of 0.51%

railroad, all in an untreated condition, are shown in Table 7. The results on the soft steel are the same. In the medium steel there is a slight falling off in the physical properties when the sulphur exceeds 0.100%. The highest carbon, giving the results on specimens taken at the

TABLE 7. TENSILE TESTS OF HIGH SULPHUR STEELS

| Carbon Content, % | Sulphur Content, % | Elastic Limit, Lb. per Sq.In. | Tensile Strength, Lb. per Sq.In. | Elongation in 8 In., % | Reduction of Area, % |
|---|---|---|---|---|---|
| | | Untreated ¾-In. Diameter Rounds | | | |
| 0.09 | 0.030 | 31,360 | 50,460 | 30.8 | 64.2 |
| 0.09 | 0.060 | 32,740 | 50,900 | 30.2 | 65.3 |
| 0.09 | 0.090 | 30,890 | 51,400 | 31.2 | 62.5 |
| 0.09 | 0.140 | 31,600 | 50,700 | 32.5 | 64.2 |
| 0.09 | 0.180 | 31,530 | 50,960 | 30.7 | 62.3 |
| | | Tensile Tests of 8-In. Channels | | | |
| 0.32 | 0.032 | 45,300 | 71,580 | 25.5 | 54.8 |
| 0.32 | 0.068 | 45,000 | 70,060 | 26.2 | 54.8 |
| 0.32 | 0.108 | 47,110 | 70,670 | 24.2 | 52.9 |
| 0.32 | 0.146 | 46,210 | 70,060 | 26.2 | 50.9 |
| 0.32 | 0.190 | 48,930 | 70,060 | 24.2 | 48.4 |
| 0.32 | 0.230 | 47,250 | 67,920 | 24.5 | 47.9 |
| | | Tesile Tests of Untreated Axles* | | | |
| 0.51 | 0.025 | 44,320 | 92,160 | 22.0 | 34.5 |
| 0.51 | 0.055 | 41,250 | 91,670 | 20.5 | 35.7 |
| 0.51 | 0.095 | 43,220 | 85,650 | 22.0 | 35.8 |
| 0.51 | 0.135 | 35,770 | 81,330 | 23.0 | 35.9 |
| 0.51 | 0.167 | 35,400 | 80,340 | 22.0 | 34.3 |
| 0.51 | 0.230 | 36,390 | 80,060 | 22.0 | 34.6 |

*Axles 4¼ in. in diameter at center. Journal 4¼ x 8 in.

edge, shows that as the sulphur rises there is a decrease in the ultimate strength but an increase in the toughness, as shown by the reduction of area. Specimens cut halfway to center of axles gave similar results.

The bending qualities of the 0.09% carbon steel is shown in the tests of rivets, the bending quality of the 0.32% carbon steel in the opening and closing tests of the 8-in. channels. Bending tests were not made on full-sized material made from the 0.51% carbon steel.

In addition to the tests mentioned above, deflection tests were made on the full-sized channels and drop tests

TABLE 8. DEFLECTION TESTS ON 8-IN. CHANNELS*

| Carbon Content, % | Sulphur Content, % | Load to Deflection, Lb. | Permanent Deflection, In. |
|---|---|---|---|
| 0.32 | 0.032 | 22,400 | 0.04 |
| 0.32 | 0.068 | 21,900 | 0.04 |
| 0.32 | 0.108 | 22,500 | 0.04 |
| 0.32 | 0.146 | 22,300 | 0.04 |
| 0.32 | 0.190 | 23,200 | 0.04 |
| 0.32 | 0.230 | 23,100 | 0.04 |

*Deflection tests made on full-sized 8-in. channels, 11.25 lb. per ft., 48 in. between centers. Load applied at center.

on the axles. The results of the deflection tests on the channels are shown in Table 8, and the results of the drop tests on axles in Table 9.

## Summary of Results Secured

In practically every case finished articles in common everyday use were made. The work was carried out by the ordinary methods practiced, and in such sizes as are manufactured in the mills or shops. A great deal of work covering other lines of manufacture has been done, and additional work is either under way or in contemplation. The results presented speak for themselves and need very little comment. The author does not advocate paying no attention whatever to sulphur content of steel

TABLE 9. DROP TESTS OF 0.51-CARBON UNTREATED AXLES*

| Sulphur Content, % | Number Blows to Break Axle | Deflection in In. After Each Blow Axles Turned Over After Every Second Blow: First Blow | Second Blow | Third Blow | Fourth Blow | Fifth Blow | Sixth Blow | Seventh Blow |
|---|---|---|---|---|---|---|---|---|
| 0.025 | 33 | 4¾ | 5¾ | 3¾ | 5¾ | 3¾ | 5¾ | 3 |
| 0.055 | 29 | 4¾ | 5¾ | 3¾ | 5¾ | 3¾ | 5¾ | 3¾ |
| 0.095 | 30 | 4¾ | 5¾ | 3¾ | 5¾ | 3¾ | 5¾ | 3¾ |
| 0.135 | 30 | 4¾ | 6 | 3¾ | 6 | 3¾ | 5¾ | 3¾ |
| 0.167 | 21 | 4¾ | 5¾ | 3¾ | 5¾ | 3¾ | 5¾ | 3¾ |
| 0.230 | 14 | 4¾ | 5¾ | 4 | 5¾ | 4 | 5¾ | 5¾ |

* Axles rough turned. Size 4¾ in. in diameter at center. Journal, 4¼x8 in. Weight of tup, 2,200 lb. Height of drop, 16 ft.

but believes firmly that a steel containing less than 0.100% is not necessarily bad, and that it will show little, if any, difference in quality when compared with the same steel of much lower sulphur, other conditions being the same.

# Track-Scale Specification

The United States Bureau of Standards has just issued in preliminary draft a specification for railroad track-scales, thus taking the first big step toward regulating conditions in a field concerning which there has been much confusion and for some years past acrimonious discussion.

The specification defines the capacity of the scale as the weight of the heaviest car the scale will weigh, provided that it will support a train of such cars passing over the live rail without stresses in excess of those specified being developed in the members of the scale.

The stresses for all working parts of the scale are given in the accompanying table; no separate allowance for impact is necessary.

| Nature of Stress | Cast Iron | Cast Steel | Wrought Iron | Structural Steel | High-Carbon Steel for Knife-Edges and Bearings |
|---|---|---|---|---|---|
| Tension | 1,500 | 8,000 | 8,000 | 10,000 | 30,000 |
| Compression | 8,000 | 10,000 | 8,000 | 10,000 | 30,000 |
| Transverse bending: | | | | | |
| Tension | 2,000 | 8,000 | 8,000 | 10,000 | 30,000 |
| Compression | 8,000 | 10,000 | 8,000 | 10,000 | 30,000 |
| Shear | 2,500 | 6,000 | .... | 7,000 | .... |
| Torsion | 2,500 | 6,000 | .... | 7,000 | .... |

Stress per lineal inch of knife-edges, 4,000 lb. Compressive strength of concrete, 400 lb. per sq.in.

A representative car for calculations is one having 5½-ft. trucks with centers 22 ft. apart and centers of adjacent end trucks of coupled cars 12 ft. apart. Concerning other points the following items may be noted: The scale bridge should be of the articulated-girder type having "either separate girders that are properly joined at the supports or continuous girders with flexible sections at each support." Knife-edges should be used and not point or button bearings. They should be firmly fastened to the levers and so mounted that uniform contact and pressure will occur between the knife-edges and the bearings, the latter condition requiring reinforcement against transverse bending. The distance between fulcrum and load knife-edges in the main levers should not be less than 10 in. Nose irons and levers should make contact by machined surfaces. Adjustment of the nose irons should be of such type that the knife-edges will remain parallel to their original position. The adjustment is best made by a bronze machine screw, with number of threads per inch stamped on the head of the screw, and clamp screws provided to hold the nose iron in position. The normal position of the nose iron should be marked by match marks on the nose iron and the lever. The poise on the beams should have a horizontal line of motion when the beam is in midposition. The play of the beam in the trig loop should be 2% of the distance from the fulcrum knife-edge to the loop. The sensibility reciprocal, or the weight required on the platform to turn the beam from the horizontal to the top of the loop, should never be greater than 100 lb., and for a new scale should not be greater than 50 lb. The main beam should be notched at every 1,000 lb., and its graduation should represent not more than 6,000 lb. per in. Graduations of the fractional beam should not represent more than 200 lb. per in. and the graduation interval should be not less than one-half the sensibility reciprocal, or generally 50 lb., except for recording beams. The indicator of the fractional poise must be symmetrical with respect to the graduation. There should be no tare beam.

# Draining the Roseau River Swamp in Minnesota

By George H. Herrold*

*SYNOPSIS—The river had a winding course with very little fall through swamp lands, beyond which it had a steeper grade. Improving the channel and equalizing the grade give good drainage. The work is in a district difficult of access.*

Drainage work now under way in Roseau County, Minnesota, provides for widening, straightening and deepening the Roseau River from Lake Roseau to the Canadian boundary (36 mi.) and for the immediate construction of lateral canals and ditches to reclaim 50,000 acres of swamp land owned by the state and 40,000 acres privately owned. This project is financed by the state and is under the jurisdiction of the State Drainage Commission.

The river flows almost directly north as far as the lake, which is a depression, or suck-hole, in the glacial drift, about 4 mi. south of the boundary. From this lake the water overflows and forms an impassable marsh covering over 300 sq.mi. The drainage area is 980 sq.mi. above the lake and 1,350 sq.mi. at the Canadian boundary.

West of the lake the river channel was almost undefined for 25 mi., its flow of 0.3 ft. per mi. not giving sufficient velocity to erode a channel. After passing the edge of the great moraine, it flows down the north slope between defined banks with a fall of 1½ ft. per mi. for 12 mi. to the boundary. Crossing at Caribou City, it continues to the Red River at Dominion City, Manitoba. The river channel and the auxiliary drainage works are shown in Fig. 1.

Development of some of the higher lands demonstrated the fertility of the soil. The state became interested through its holdings and investigated the possibilities of drainage. The examination resulted in dredging a small channel for the river from Roseau Lake west across the swamp to the steeper reach of the river (Fig. 4) and also in building laterals to drain land lying to the south.

The river channel was kept open by redredging for several years with little result, but the slight lowering of the water level made possible more careful exploration and survey. It was shown that the swamp lands were slightly lower than the dredged channel, that the fall in 24 mi. was only 0.3 ft. per mi., and that the 18-ft. fall of the river in the last 12 mi. within the United States must be distributed over the 24 mi. lying east to Roseau Lake. It was shown also that the entire country is underlain with hardpan of unknown thickness.

In 1913 the legislature appropriated $80,000 to improve the river and to provide an outlet for the drainage system. Surveys and estimates showed that this amount was sufficient to make a channel for 17 mi. east from the Canadian boundary. The estimate was based on a channel with a 60-ft. concave bottom and a grade line that would increase the fall through the marsh from 0.3 to 0.7 ft. per mi. This effect was accomplished by lowering the rived bed through the hardpan stratum underlying it in the west edge of the marsh. Cutoffs across long winding loops reduce the distance to a minimum and utilize the available fall to the best advantage.

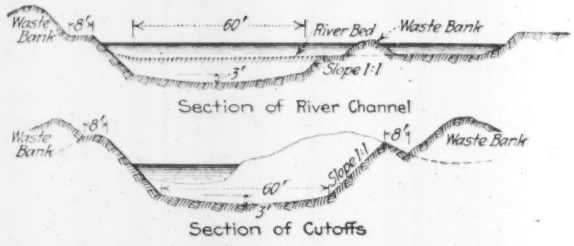



FIG. 2. CROSS-SECTION OF DREDGED CHANNELS

The distribution of cost is as follows: 450,154 cu.yd. of earth excavation at 10c., $45,105; 84,038 cu.yd. of hardpan at 25c., $21,010; two timber bridges, $3,000; moving one bridge, $500; removing sand bars, $700; right-of-way, damages due to cutoffs, etc., $2,500; surveys and supervision, $7,185; total, $80,000.

### Dredging the River Channel

A contract was let to the Central Dredging Co., of Cleveland, Ohio, on a yardage basis. Work was started in August, 1914, and the 17-mi. channel is nearly completed. An additional appropriation of $55,000 has been made to complete the work to Roseau Lake, and the company will continue this in 1916. The plans for the second contract provide for diverting the river, before it enters the lake, by means of a 50-ft. channel gradually

*City Engineer's office, St. Paul, Minn.

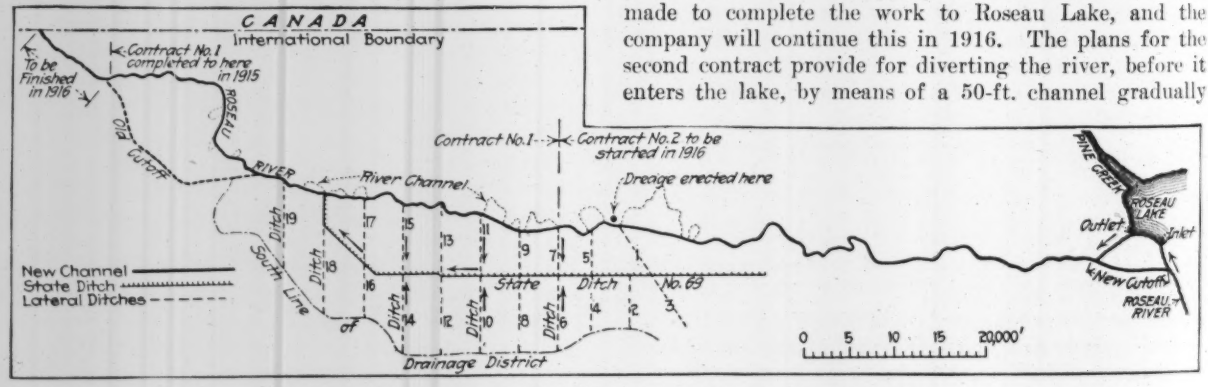



FIG. 1. THE ROSEAU RIVER DRAINAGE SYSTEM IN MINNESOTA

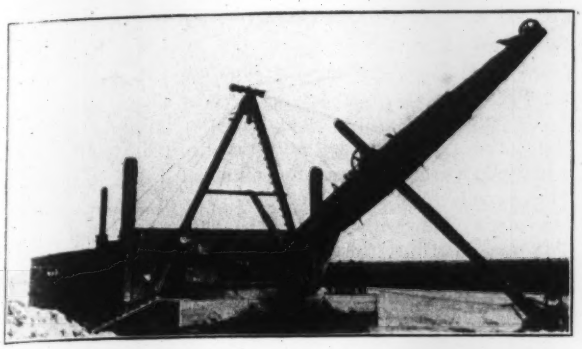


FIG. 3. DITCHING DREDGE ON THE ROSEAU RIVER CHANNEL IN MINNESOTA

widening to 60 ft. at the beginning of the first contract (Fig. 1).

Fig. 2 shows cross-sections of the main river channel and the cutoffs. The right-of-way for the former extends 33 ft. from the edge of each bank; on the cutoffs it is 200 ft. wide. Contracts were let also for state ditches Nos. 69 and 72, with laterals on all section lines. Ditch No. 72 is outside the southern limits of the district. The lateral ditches are 3 ft. deep at the summit and have a fall of 1 ft. per mi.

The river work was done with a Fairbanks dredge, shown in Fig. 3. This machine has a hull 40x90 ft., with vertical anchor spuds. It has a 72-ft. boom and a 2-yd. dipper of manganese steel fitted with teeth for the hardpan excavation. The maximum monthly yardage was 80,000 cu.yd.

With the great distance from railways and the lack of wagon roads it is difficult to obtain coal and camp supplies. The general method is to haul the coal over the ice during the winter and distribute it as nearly as possible where it will be wanted the coming season, but much coal distributed in this way was never found after the spring thaw. The mosquitoes constitute a plague during the working season.

At Roseau, Minn., a number of flat-bottom scows of 18-in. draft were built, and on these the dredging machinery, material and equipment were towed by a stern-wheel gasoline launch. The 72-ft. boom had a maximum depth of 7 ft. It was loaded on two scows, and the bridges along the route had to be cut, as they were too low for this clearance. Coal was taken in the same manner for the beginning of operations in 1914. The dredging outfit was unloaded at a loop in the river, as shown in Fig. 1; after being assembled, it cut its way through to the beginning of the contract. In the spring of 1915 the base for supplies was Ross, Minn. Later it was changed to Colienta, Manitoba, supplies being shipped in bond to this point and then hauled to the river by teams.

### Good Results of Land Drainage

The results are far exceeding expectations. Already gas tractors hauling gang plows have been breaking up large tracts of land which a few years ago could not be crossed with a team and were dangerous to cross on foot. The cost to the state of draining its land is estimated at less than $3 per acre, and this land will sell for from $10 to $20 per acre. Lands privately owned which are benefited by this project are assessed their pro rata cost per acre benefited.

The adequacy of this improvement, from an engineering standpoint, is a question of considerable interest. The maximum flow is estimated at 1,200 sec.-ft., or 0.9 sec.-ft. per sq.mi. of drainage area, which is equivalent to 12 in. of rainfall. The average rainfall is 20 in., with a range of 16 to 32 in. The mean temperature is 35° F. The carrying capacity of the new river channel, using the cross-section shown and 8-ft. depth of water, is 1,046 sec.-ft., or less than the flood flow under conditions prior to the making of the improvement.

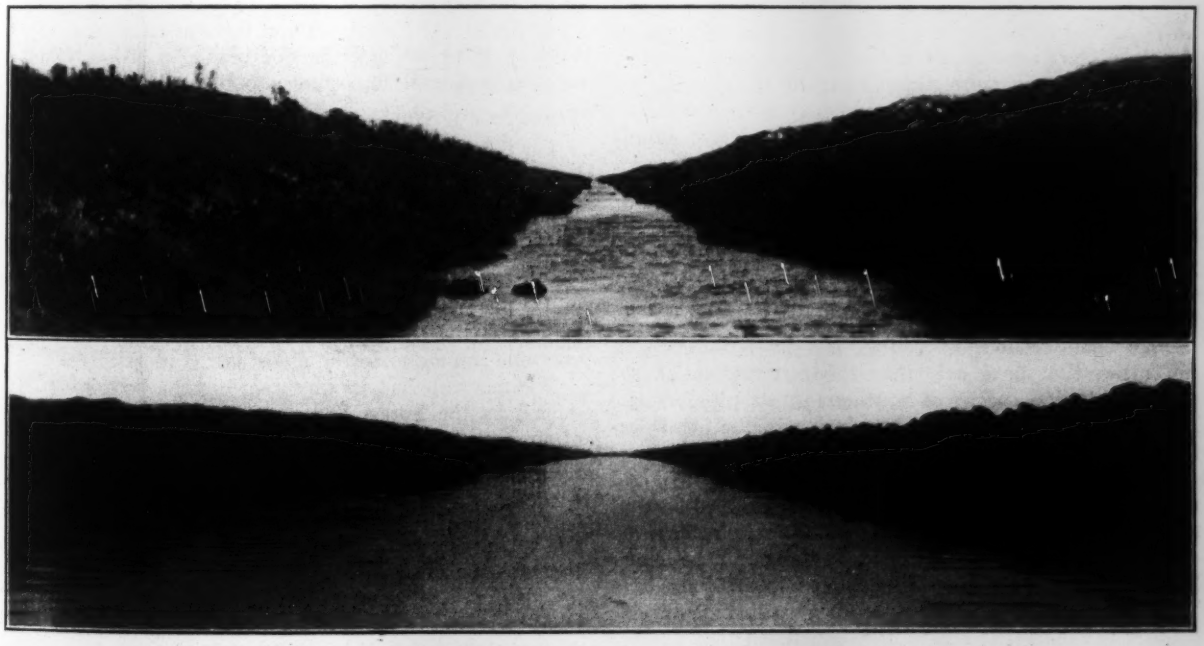


FIG. 4. THE ROSEAU RIVER CHANNEL ACROSS THE SWAMP LANDS OF MINNESOTA
Above is the first dredged channel. Below is the channel as now improved

The swamp has acted as a regulator of the flow, retarding it in flood times and giving up its waters in dry periods. Therefore the maximum estimated run-off at the boundary line cannot be considered as the flood flow after the river has been widened and deepened through the swamp and laterals have been constructed on every section line. Fig. 4 shows a comparison of the first dredged channel of the river (previously mentioned) and the present improved channel.

It is to be expected that during the draining of this area there will be a period of flooding of the country in the vicinity of the boundary. But after the swamp is drained, which will occur in about two years (if there is not a period of exceptional rainfall), the conditions will be of more normal character, with cultivated fields and growing crops reducing the run-off. Except for great flood periods, the new channel will be of ample dimensions.

An important improvement is the building of roads along the river channel and main ditches under state contracts, using the excavated material in the spoil-banks. These roads have helped wonderfully in opening up the territory. They are finished to a top width of 16 ft. The specifications for all ditches, except those of great width, require that the waste bank must be placed on one side, and in all cases require it to be leveled off and dragged to form a roadway. Culverts 4 to 6 ft. in diameter, of corrugated iron, carry laterals under roads.

# Yellow-Pine Timber Graded Without Guesswork

By Hermann von Schrenk*

*SYNOPSIS—Erroneous views about yellow-pine supply and quality arose from inability to grade the timber by botanical classification. Tests showed that strength did not depend on botanical species. By substituting density as the criterion, a valuable and easily applied rule for grading was formulated and made standard. Every user of yellow pine should familiarize himself with the rule.*

Not many months ago I heard an Eastern architect make the remark, "It no longer is possible to obtain Southern yellow-pine timbers suitable for heavy construction."

That to me was an amazing statement, coming as it did from a man whose business it was to know of the available supply of materials necessary to his work; and I was further dumfounded to notice that the other professional builders present did not contradict the statement. Evidently there was something radically wrong with the information possessed by that architect and his associates.

I happen to know that there is today a larger supply than ever before of perfectly manufactured high-grade dense Southern yellow-pine timber. That engineers and architects everywhere should not know this fact is a misfortune, in view of the great value of yellow-pine timbers for so many architectural and engineering constructions.

## Species of Yellow Pine Hard to Distinguish

The whole trouble, of course, has its foundation in difficulties that have arisen in the distribution of the various botanical species of Southern yellow pine. There has always been more or less confusion in the identification of species and in the selection of structural timbers that shall have the dense, durable character required for particular uses. Possibly it is this difficulty that created the belief of a few persons that satisfactory and dependable timbers of Southern yellow pine can no longer be obtained. Some architects and engineers have been led to believe that the supply of heavy, strong, durable timbers is exhausted; others have concluded that unscrupulous manufacturers and dealers find it to their advantage to substitute inferior timbers for those specified.

As a matter of plain fact, the trouble has been due principally, if not altogether, to the lack of any standard by which the quality of individual timbers could be determined accurately. It is hard to believe that a manufacturer or a dealer would be so shortsighted as to destroy the market for his high-grade structural material by substituting materials that fail under stress. But when we remember that until very recently even the scientists were unable to lay down a hard and fast rule by which the manufacturer, the dealer or the specification writer could measure quality in Southern yellow pine, it is not remarkable that the results of haphazard hit-or-miss delivery or acceptance led on occasion to disappointment and failure.

It is in this connection that a recent rule evolved jointly by the National Forest Service, the American Society for Testing Materials and the Southern Pine Association is of such profound importance to the building world.

## Tests Show Strength Does Not Depend on Species

For the first time since engineers and structural experts learned that dense, sound Southern yellow-pine timbers were peculiarly suited for certain uses it is now possible under this rule for any person to determine, absolutely and easily, exactly what may be expected in point of strength from every timber that goes into a structure under his supervision. It is hardly necessary for me to emphasize the tremendous value of this working rule to everyone interested in the use of structural timbers.

The new rule had its inception in exhaustive tests conducted by the Forest Service to determine the relations of strength to density in Southern yellow-pine timbers of the various species. These tests proved that strength is wholly determined by density—the closeness of annual rings and the sharpness of definition between summer wood and spring wood in the cross-section of the individual timber. They also showed that it is entirely possible for true long-leaf yellow pine to vary widely in growth structure, in some cases the rings of summer wood being so widely separated that the stick could not be depended upon for strength. On the other hand, it was found that individual

*Tower Grove and Flad Aves., St. Louis, Mo.

pieces of short-leaf or loblolly pine might have grown under such conditions as to possess the density necessary to give them rating with first-class dense long-leaf pieces.

In other words, the Forest Service experiments showed that the strength of Southern yellow-pine timber is not dependent necessarily on its botanical species—a finding that was revolutionary in view of all precedent, which had specified long-leaf yellow pine for timbers required for exceptional uses.

### Early Proposals to Classify by Density

While the Forest Service was engaged upon this experimental work, the American Society for Testing Materials was active in the same direction. The society began a series of investigations in 1909, and a committee of the society at that time pointed out that in practice the separation of the various species of yellow-pine timbers after manufacture is extremely difficult, and in many cases practically impossible. In other words, it was easy enough to specify that all timbers should be manufactured from strictly long-leaf Southern yellow pine, but it was in many cases impossible to determine whether the timbers actually received were really long-leaf or some other species. The committee also suggested that, whether they were long-leaf or not, the difference in strength in the various pieces was probably due to a very large extent to the different percentages of wood fiber per cubic foot—that is, the different densities of the various pieces.

With this in mind the committee formulated a tentative rule, suggesting the advisability of dropping the botanical designations and classifying the Southern pines into two groups, the basis of which classification was to be the number of annual rings per inch, as counted on a cross-section

A suggestion was made by A. O. Elzner, representing the American Institute of Architects, that it might be just as well to drop the terms long-leaf, short-leaf and loblolly altogether and call all Southern yellow-pine timbers Southern yellow pine, to be divided into grades 1, 2 and 3, according to the number of annual rings per inch.

### Formulating the Density Rule

To shorten the story of a long and laborious investigation on the part of the Forest Service and of the American Society of Testing Materials, the former suggested a rule using the rate of growth, the percentage of summer wood and a distinct line of demarcation of spring and summer wood as the basis for the new classification.

The American Society for Testing Materials was put in possession of all these data; and after studying them thoroughly the committee recommended the adoption with only slight modification of the rule formulated by the Government. In this form the rule was unanimously adopted by the society in August of last year.

Awake to the necessity of such a rule and recognizing the value and dependability of the findings of the Forest Service and of the American Society for Testing Materials, the Southern Pine Association, an organization made up of manufacturers of Southern yellow-pine timbers, promptly indorsed the rule and adopted it.

The rule is characterized by shortness and simplicity:

**Dense Southern yellow pine shall show on either end an average of at least six annual rings per inch and at least one-third summer wood, or else the greater number of the rings shall show at least one-third summer wood, all as measured over the third, fourth and fifth inches on a radial line from the pith. Wide-ringed material excluded by this rule will be acceptable provided that the amount of summer wood as above measured shall be at least one-half.**

**The contrast in color between summer wood and spring wood shall be sharp, and the summer wood shall be dark in color, except in pieces having considerably above the minimum requirement for summer wood.**

**In cases where timbers do not contain the pith and it is impossible to locate it with any degree of accuracy the same inspection shall be made over 3 in. on an approximate radial line beginning at the edge nearest the pith in timbers over 3 in. in thickness and on the second inch (on the piece) nearest to the pith in timbers 3 in. or less in thickness.**

**In dimension material containing the pith, but not a 5-in. radial line, which is less than 2x8 in. in section or less than 8 in. in width, that does not show over 16 sq.in. on the cross-section, the inspection shall apply to the second inch from the pith. In larger material that does not show a 5-in. radial line the inspection shall apply to the 3 in. farthest from the pith.**

**Sound Southern yellow pine shall include pieces of Southern pine without any ring or summer-wood requirement.**

### Merits of the Density Rule

While the new rule may at first sight appear to be a somewhat radical departure from past standards, a careful study will show that such is not really the case. The density rule, when applied to a mixed lot of the various Southern pine timbers of the several botanical species, will include most of the pieces of the true botanical long-leaf pine in the grade hereafter to be known as "dense pine"; a smaller percentage of the denser pieces of loblolly, Cuban and short-leaf pine also will fall within the dense grade; and on the other hand, a small percentage of the more rapid-growing pieces of long-leaf will be excluded, as will also a great majority of pieces of loblolly, Cuban and short-leaf.

It should be clearly understood that the classes "dense pine" and the less desirable "sound pine" refer specifically to quality of density when considered from the structural or strength standpoint and that they replace the botanical terms hitherto used—long-leaf, short-leaf, loblolly, etc.

It should also be clearly understood that the usual specifications as to the percentage of heart and sap are in no way changed. In other words, where timber for long service is demanded, together with high strength qualities, dense pine should be specified with a minimum amount of sapwood, or what is known as heart timber.

A feature of the new rule that particularly recommends it is that it is easy of application. For the first time there is available for classifying structural timbers a rule which is based on actual measurement and which has nothing to do with catch-judgment or feeling. Numerous diagrams and illustrations in the Southern Pine Association's density-rule book show how simple is the method and how effectually disagreements are eliminated.

To promote the practice of standardizing high-grade dense timbers, the association just mentioned has adopted the policy of branding each individual timber. When this practice has been generally adopted, it will be a further guarantee of quality, serving to make assurance doubly sure to the builder having occasion to use yellow pine.

It seems to me that every architect and engineer in America should make it his business immediately to familiarize himself with this new grading rule and the method of its application. Since yellow pine has qualities that make it most desirable for certain uses, it is a matter for congratulation to manufacturers and users that the uncertainties have been eliminated in specifying the material in heavy construction. This is the more true because, as already stated, ample supplies of yellow pine are available, opinion to the contrary notwithstanding.

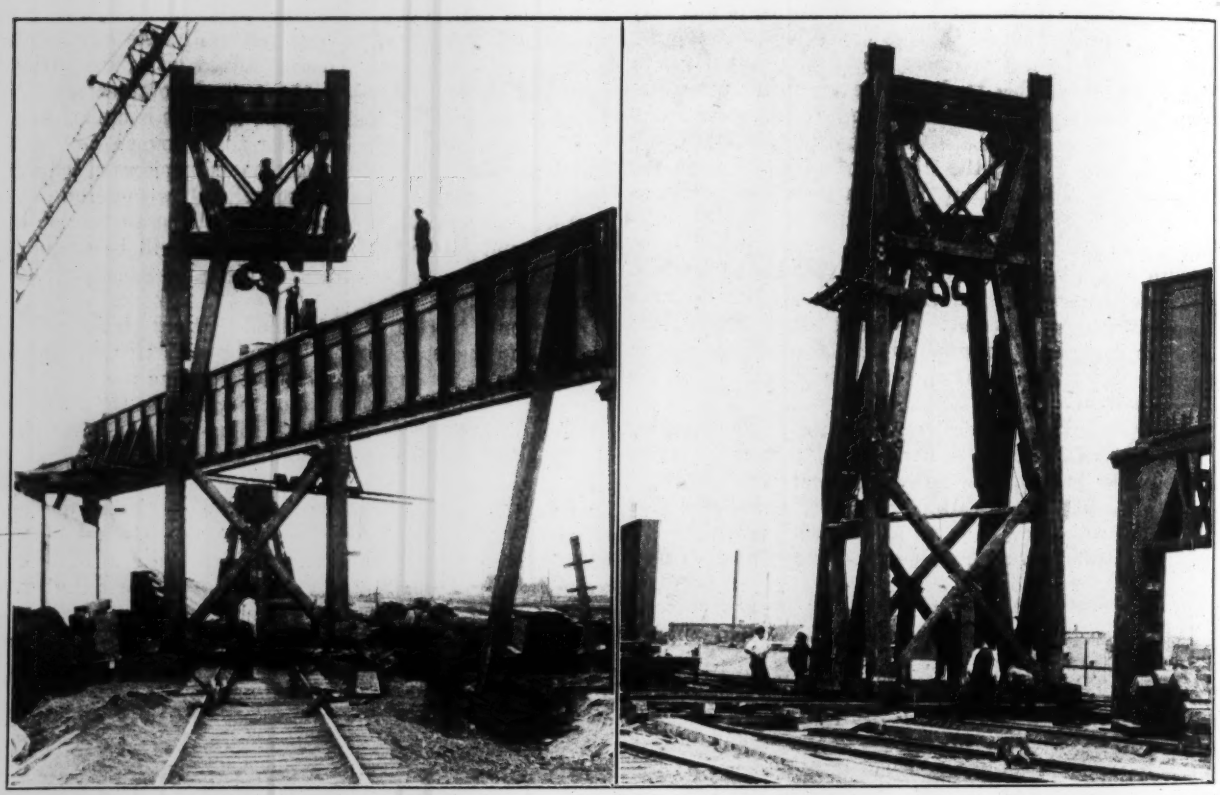
# Field and Office

## Erecting Long Plate Girders by Tower Traveler

To erect the record-breaking plate girders of the five-span Nickel Plate crossing over the Illinois Central R.R. tracks at 79th St., Chicago, required a plant and method capable of quick work, to avoid blocking the traffic on the Illinois Central tracks below. The bridge itself and the heavy girders composing its five spans were described and illustrated by drawings in *Engineering News* of Dec. 9, 1915, p. 1105. The heaviest girders weigh 130½ tons and are 125 ft. 8 1/16 in. long; the longest girders are 131 ft. 9 in. over all.

All girders were received on cars on the Illinois Central tracks and were hoisted up to place by a four-leg tower that straddled the girders being lifted. Removable leg sections in the rear bent of the tower provided for disengaging it from a girder just erected and moving it forward to the next. The tower was slid from girder to girder on the track rails. In sliding it over to the next span, transverse rails laid on the track rails served as bed. The group of views herewith shows some of the typical stages in the erection work and illustrates the general procedure clearly.

Figs. 1 and 2 show the tower and the method of moving. How it was used can be seen best from the large view, Fig. 4. Straddling one of the Illinois Central tracks, the tower picked up a girder brought in on cars on this track. A set of hooks attached to a loading beam suspended in the tower engaged the girder at the middle (the girders sometimes had to be balanced by iron piled on one end). Then the girder was hoisted to the level of its final position, Fig. 3, swung around to the alignment of the bridge and set on its supports, Fig. 4. The first girder, Fig. 1, when set, was strutted at both ends, besides being held by the attachment of floor-beams at the abutment.



FIGS. 1 AND 2. THE A-FRAME AND TOWER USED FOR ERECTING THE NICKEL-PLATE GIRDERS AT 79TH ST., CHICAGO—METHOD OF MOVING ALONG AND ACROSS TRACKS

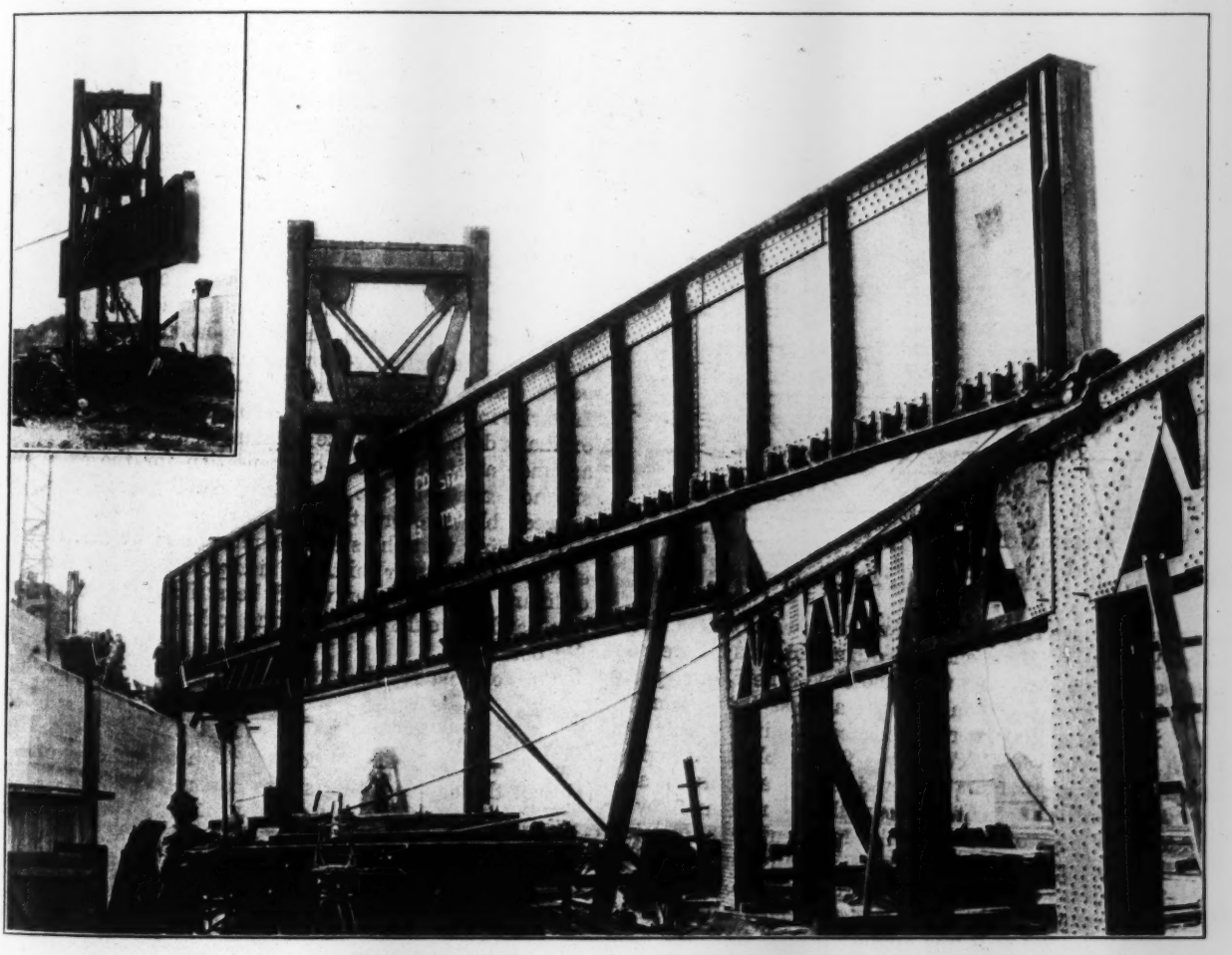
Fig. 1—First girder set on final bearings and braced by floor-beams and struts; lower panel of cross-bracing inserted in tower, and "gate" section of posts on one side removed; tower just being hauled forward to location of next girder. Fig. 2—Sliding the tower to the next span on rails laid over the Illinois Central R.R. rails

The condition of the tower in Fig. 1 is that required for moving forward along the track to the second girder. Crossbracing is inserted in the lower panel of the tower, and in the panel next above, the rear legs are removed, leaving the top frame of the tower cantilevered from the other pair of legs. Thus the tower can be slid forward clear of the girder it has just set and ready for reassembling to take hold of another girder.

FIG. 5. LAST GIRDER OF BRIDGE SET IN PLACE OCT. 29, 1915

The tower had to be placed quite accurately, as the girder, when swung, could be moved only a trifle endwise and practically nothing laterally. This limitation meant also that the tower had to be set level. In spite of the care required in this work, the erection attained a speed of one girder every other day, after the gang got fairly under way. The first of the 15 girders was set Sept. 15, and the last on Oct. 29, 1915.

In preparing for the erection the field party located the bearings to the precise spacings of the girder shoes, as given by shop measurement after fabrication. As a result, no bearing missed connection by more than $\frac{1}{16}$ in.



FIGS. 3 AND 4. METHOD OF HOISTING AND SETTING GIRDERS WITH THE TOWER

Fig. 3, Sept. 15, 1915, shows the first girder of the bridge being lifted from cars, ready to swing around into place. In Fig. 4 the second girder is just being landed on its bearings; compare with Fig. 1 to note condition of tower for moving and for hoisting

It must be remembered that the bridge is on a very sharp skew (about 20° to Illinois Central tracks), which increases the difficulties of accurate setting. The two girders meeting on a bent have a joint base-plate, with integral half-pin seats for notches in the girder shoes.

The bridge was erected for the New York, Chicago & St. Louis R.R., of which George H. Tinker is Bridge Engineer; A. C. Harvey was Field Engineer. The work was done by the Pennsylvania Steel Co., Steelton, Penn., under J. F. Ritter, Superintendent of Erection.

## Thrust of Steel Arch Adjusted by End Jacking

A railway street crossing over Markt St., Berlin, Germany, which has its girders and columns formed in one piece, acting as an arch structure, was adjusted in the course of erection to precise theoretical action by horizontal jacking. R. Kramer describes this process in *Der Eisenbau* of September, 1915. In earlier cases of jacking two-hinged arches the force was applied at the crown. Jacking at the abutment is a novel procedure.

The bridge has a span of 80 ft. and comprises seven parallel girders or frames. Each girder was erected on

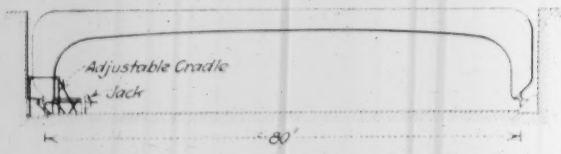



FIG. 1. SKETCH OF ARCH STREET CROSSING, CRADLE AND HORIZONTAL JACKING ARRANGEMENT

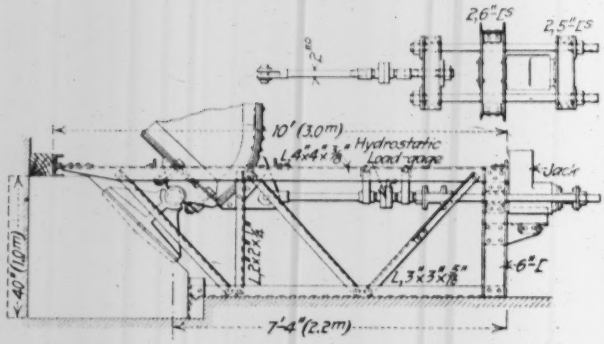



FIG. 2. HORIZONTAL JACKING DEVICE

falsework above true position and then was lowered upon a rocker cradle at one end, the other end being seated on its permanent shoe. At the same time a horizontal jacking frame was placed in position against the abutment, and its pulling rods were connected to hitches near the base of the arch leg. Force was first applied through a hydraulic jack, but later a screw jack was substituted and proved more convenient. The pull was measured by a 20-ton load gage of the water-displacement type, inserted in the pulling rod. The thrusts actually put into the several arches, determined in advance by computations from the known weights, etc., ranged from 10 to 13 tons. When the desired pull was applied by the jack, the shoe was put in place against the pin of the arch and blocked and grouted in position, thus holding the arch in the precise position required to give the correct thrust.

## Steam-Hammer Pile Formulas; Suggestions and Queries

By A. M. Knowles*

I read with much interest the article entitled "Test Comparing Steam and Drop Hammer Pile Formulas," *Engineering News*, Jan. 6, 1916. This article is of particular interest, as there has been so little material available concerning the actual relative performance of ordinary drop hammers and gravity-drop steam hammers, the latter being (I assume from its description) the type of steam hammer used at the Pearl Harbor and Mare Island dry docks. There seems to be still less reliable information available concerning the performance of the double-acting steam hammer, which has come into quite general use, as compared with either of the two other types of hammer.

While the theoretical part of the *Engineering News* formulas is generally accepted, the results of the formulas depend to a great extent on the constant used in the denominator, which can only be determined with any degree of accuracy by actual comparisons of tests such as Mr. Gayler has made. It does not seem possible that this very important field of engineering has really been neglected to the extent that would appear from any literature there is available. Undoubtedly there are engineers who have conducted tests along these lines with the view of developing formulas that will give reasonably close results when applied to piles driven under the same general conditions with each of the three types of hammers referred to. The writer, at least, and probably others would be pleased to hear of such experience through the columns of *Engineering News*.

The article referred to states: "At Pearl Harbor the average penetrations per blow by steam hammer and by drop hammer at the same attained depth were almost exactly proportional to the respective energies of the blow." From this comparison would it not be correct to assume that the constant in the denominator of the generally accepted gravity-drop steam-hammer formula depends almost entirely upon the interval between blows? Likewise in the original *Engineering News* formula for the ordinary drop hammer the constant 1 was, I understand, assumed arbitrarily as a compensation for the extra resistance caused by the settling of earth around the pile between blows, over other ordinary resistances.

Assuming that the steam hammer used by Mr. Gayler struck 60 blows per min. and using his constant 0.3, the ordinary drop hammer would be striking 18 blows per min., and it would be interesting to know if this relation did obtain.

The formula for the gravity hammer

$$P = \frac{2Wh}{s+1}$$

may be written in the following form to make it applicable to the double-acting steam hammer:

$$P = \frac{2h(Am + W - b)}{s+k}$$

in which

$P$ = Safe bearing load, pounds;
$h$ = Effective length of stroke, feet;
$A$ = Effective area of piston, square inches;

*Assistant Engineer of Bridges and Buildings, Erie R.R., 50 Church St., New York.

$m$ = Mean effective pressure, pounds per square inch;
$W$ = Weight of moving part, pounds;
$b$ = Total back pressure;
$s$ = Penetration under last blow, inches;
$k$ = Constant.

The uncertain values in this formula are $m$, $b$ and $k$.

A manufacturer's catalog in commenting on the advantage of a double-acting steam hammer over a gravity steam hammer states: "The free fall of the hammer being accelerated by the steam pressure, the number of blows per minute is increased while the force of the blow is maintained." Therefore, with the force of the blow maintained, the constant should be decreased in proportion to the number of blows struck per minute, and assuming 60 blows per min. for a gravity-drop steam hammer and 100 blows per min. for a double-acting steam hammer, both hammers doing the same work per blow, and using the constant 0.3 as determined in the *Engineering News* article for the gravity-drop steam hammer, would not the constant for the double-acting steam hammer be in the proportion of $\frac{60 \times 0.3}{100}$, or 0.18? This would naturally give much higher bearing value for equal penetration per blow.

For the double-acting steam hammer the total force upon the ram consists of its weight and the steam pressure on the cylinder, which is usually assumed as the mean effective pressure for the entire length of the stroke with a suitable boiler pressure. This mean effective pressure varies naturally as the throttle lever is changed to regulate the speed of the hammer, to say nothing of the variation of the boiler pressure; and so, would not the constant depend upon the man at the throttle lever? Also, from actual experiments with indicator diagrams it has been found that the mean effective pressures so assumed are erroneous and much higher than found in actual practice. It would then seem that a suitable formula that could be used with the double-acting steam hammer is still more variable and complex than for the other two types of hammers and should certainly be given due consideration.

## Some Better Kutter's Formula Coefficients

By Robert E. Horton*

The ordinary category of coefficients of roughness for use in the Kutter formula (which may be found in various books on hydraulic calculations) lists, as a rule, not more than 12 to 25 classes of pipes, conduits and channels, with one coefficient of roughness for each. This number of classes of channels is too limited; and the number of values of the coefficient and the range of value are not sufficient to cover modern requirements in calculation. Furthermore, various types of channels and conduits now frequently used are not listed at all in the older schedules.

It is well known that to select the proper coefficient of roughness to be applied under given conditions in connection with the Kutter formula requires considerable experience and judgment. The working engineer is loath to spend his time hunting through original sources of data and experiments, and the range of intermediate values between those given in the existing schedules is so wide as to leave large room for error or diversity of opinion.

Many values of the coefficient of roughness for new conditions have been recently worked out and published. These newer values relate to flow in concrete pipes and conduits; irrigation ditches, canals and flumes; circular, semicircular and other steel pipes and channels; and natural river channels.

In order to facilitate the rapid selection of satisfactory values of the coefficient of roughness and to standardize office-engineering practice in such calculations, the writer has made out the accompanying table of coefficients. In this table four coefficients of roughness are given for each class of channel or conduit, the intention being to cover the usual range of values occurring in actual experience and also to include the values that may be expected to apply (1) to new work, in the best order and condition, and (2) to pipes or channels that have been in use for a greater or less length of time.

Obviously it is necessary to use different values of the coefficient of roughness for the same channel, according to the conditions to be met. For example, if the thing to be determined is the actual quantity of water that a pipe will carry when new, then the coefficient for the pipe in perfect order should be used. However, if as is often the case in designing, it is desired to provide pipe or conduit capacity sufficient to convey a certain amount of water when the pipe or conduit has been in use for a term of years, a higher coefficient of roughness will in general be required.

Existing schedules of coefficients of roughness do not, as a rule, contain any values exceeding about 0.040, which is the highest value commonly given for natural river

NEW TABLE OF n FOR KUTTER'S FORMULA (R. E. Horton)

| Surface | Perfect | Good | Fair | Bad |
|---|---|---|---|---|
| Uncoated c.-i. pipe | 0.012 | 0.013 | 0.014 | 0.015 |
| Coated c.-i. pipe | 0.011 | 0.012* | 0.013* | |
| Commercial w.-i. pipe, black | 0.012 | 0.013 | 0.014 | 0.015 |
| Commercial w.-i. pipe, galv. | 0.013 | 0.014 | 0.015 | 0.017 |
| Smooth brass and glass pipe | 0.009 | 0.010 | 0.011 | 0.013 |
| Smooth lockbar and welded "OD" pipe | 0.010 | 0.011* | 0.013* | |
| Riveted and spiral steel pipe | 0.013 | 0.015* | 0.017* | |
| Vitrified sewer pipe | 0.010<br>0.011 | 0.013* | 0.015 | 0.017 |
| Glazed brickwork | 0.011 | 0.012 | 0.013* | 0.015 |
| Brick in cement mortar, brick sewers | 0.012 | 0.013 | 0.015* | 0.017 |
| Neat cement surfaces | 0.010 | 0.011 | 0.012 | 0.013 |
| Cement mortar surfaces | 0.011 | 0.012 | 0.013* | 0.015 |
| Concrete pipe | 0.012 | 0.013 | 0.015 | 0.016 |
| Wood-stave pipe | 0.010 | 0.011 | 0.012 | 0.013 |
| Plank Flumes: | | | | |
| Planed | 0.010 | 0.012* | 0.013 | 0.014 |
| Unplaned | 0.011 | 0.013* | 0.014 | 0.015 |
| With battens | 0.012 | 0.015* | 0.016 | |
| Concrete-lined channels | 0.012 | 0.014* | 0.016* | 0.018 |
| Cement-rubble surface | 0.017 | 0.020 | 0.025 | 0.030 |
| Dry-rubble surface | 0.025 | 0.030 | 0.033 | 0.035 |
| Dressed-ashlar surface | 0.013 | 0.014 | 0.015 | 0.017 |
| Semicircular metal flumes, smooth | 0.011 | 0.012 | 0.013 | 0.015 |
| Semicircular metal flumes, corrugated | 0.0225 | 0.025 | 0.0275 | 0.030 |
| Canals and Ditches: | | | | |
| Earth, straight and uniform | 0.017 | 0.020 | 0.0225* | 0.025 |
| Rock cuts, smooth and uniform | 0.025 | 0.030 | 0.033* | 0.035 |
| Rock cuts, jagged and irregular | 0.035 | 0.040 | 0.045 | |
| Winding sluggish canals | 0.0225 | 0.025* | 0.0275 | 0.030 |
| Dredged earth channels | 0.025 | 0.0275* | 0.030 | 0.033 |
| Canals with rough stony beds, weeds on earth banks | 0.025 | 0.030 | 0.035* | 0.040 |
| Earth bottom, rubble sides | 0.028 | 0.030* | 0.033* | 0.035 |
| Natural Stream Channels: | | | | |
| (1) Clean, straight bank, full stage, no rifts or deep pools | 0.025 | 0.0275 | 0.030 | 0.033 |
| (2) Same as (1), but some weeds and stones | 0.030 | 0.033 | 0.035 | 0.040 |
| (3) Winding, some pools and shoals, clean | 0.035 | 0.040 | 0.045 | .050 |
| (4) Same as (3), lower stages, more ineffective slope and sections | 0.040 | 0.045 | 0.050 | 0.055 |
| (5) Same as (3), some weeds and stones | 0.033 | 0.035 | 0.040 | 0.045 |
| (6) Same as (4), stony sections | 0.045 | 0.050 | 0.055 | 0.060 |
| (7) Sluggish river reaches, rather weedy or with very deep pools | 0.050 | 0.060 | 0.070 | 0.080 |
| (8) Very weedy reaches | 0.075 | 0.100 | 0.125 | 0.150 |

* Values commonly used in designing.

*Consulting Hydraulic Engineer, 57 North Pine Ave., Albany, N. Y.

channels containing stones or weeds. In the experience of the writer, and that of other engineers, coefficients of roughness for natural river channels often run much higher than 0.040 and are not uniform for the same channel at different stages of the stream. A natural river channel that has, for example, a coefficient of roughness of 0.035 in a certain reach at bank-full stage may at lower stages have a coefficient of roughness as great as 0.060 or even higher. This is owing, in general, to the fact that in determining the coefficients of roughness in natural river channels the entire cross-section as determined by soundings is taken into account, whereas if the cross-section is irregular there may be pockets or holes in the bed of the stream containing slack water at lower stages, so that the full area of section is not effective. Similarly at low stages of a stream there is likely to be more or less fall that is not effective as slope and that does not contribute to producing velocity.

Coefficients for natural river channels given in the table have been derived largely from unpublished experiments of the writer on the Mohawk, Black and Hudson Rivers in the State of New York. They have been confirmed by similar work of other engineers elsewhere. The matter of variation of the coefficient of roughness is more fully discussed in an article by the writer in *Engineering News*, Apr. 29, 1915, p. 827.

Experience shows that the coefficient of roughness for excavated channels of the same slope and cross-section is dependent upon the method of excavation. Additional experiments are needed to determine the specific coefficients for channels in earth excavated by different methods of dredging, etc.

More detailed lists of coefficients of roughness derived from recent experiments and utilized in preparing the small table here given may be found in "Report of the Florida Everglades Engineering Commission," Bulletin No. 194, United States Department of Agriculture; "Flow of Water in Irrigation Channels," by Fred C. Scobey, Bulletin 194, Colorado Agricultural College Experiment Station; "Frictional Resistance of Waterways," by Cohn, Trimble and Jones; and an article by Samuel Fortier, in *Engineering and Contracting*, Jan. 27, 1915.

## Drilling Methods in Driving 6-Ft. Tunnel in Granite

By S. W. Symons*

One element of the new sewage-disposal system for Portchester, N. Y., is a tunnel 900 ft. long and 5x6 ft. in cross-section, in granite. In driving the tunnel, hand hammer drills were employed, mounted on the crossarms of a center column, as illustrated in Fig. 1. The drills were cleaned by water from the city mains. The advance averaged 200 ft. per month of 25 working days, the record month being 210.5 ft.

The contract for the tunnel was let to the Daly & Merritt Co., of Portchester. W. C. Brennan was Superintendent for the contractors and was responsible for the make-up of his organization and for the methods used in driving the tunnel. Borings were made along the line, and early in April, 1915, ground was broken with steam drills at the extreme southern point. The shaft was completed and the heading started on May 8, 1915, when the steam drills were discarded and mounted "Jackhamers" were substituted.

Air was furnished by a 12x12x16-in. Rand straight-line compressor taking steam from a 50-hp. portable Ames boiler. Steam pressure was 90 lb., the air pressure 80 lb. The air was led through a 1½-in. pipe a distance of 60 ft. to the shaft and thence to the heading. The capacity of the compressor was nearly double that required by the drills, with the result that it was run most of the time at only 88 r.p.m.—a little over half its rated speed.

At the time of the writer's visit the tunnel had progressed 530 ft. northward, at which point soft ground was encountered; and it was decided to move the entire plant to the north end of the contract and drive back, in order to ascertain the exact extent of the soft material.

Fig. 2 is a diagram of the round. No. 1 drill, on the upper arm, puts in holes 1 to 9 in the upper half of the

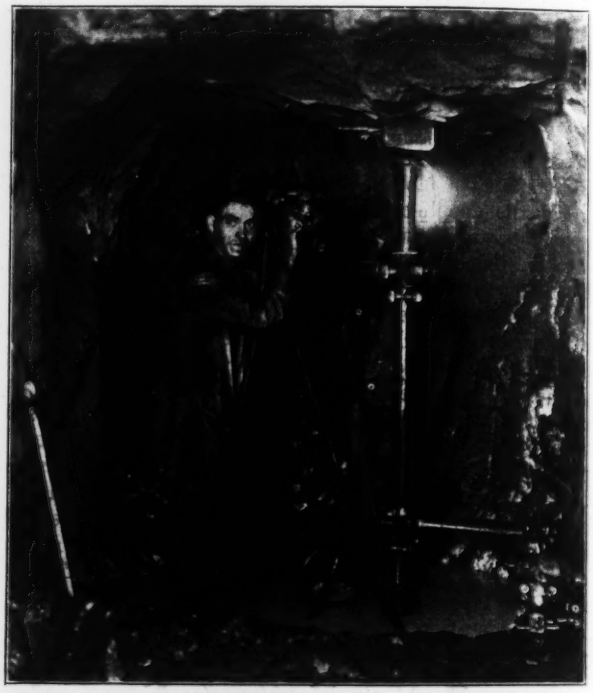


FIG. 1. HEADING OF PORTCHESTER SEWER TUNNEL, SHOWING MOUNTING OF DRILLS

heading, and No. 2, on the lower arm, drills holes 1-A to 8-A. No. 2 drill, as a rule, has one less hole to drill than No. 1, owing to a more awkward position. Two drillrunners, a nipper and two muckers constituted the tunnel crew for each shift.

Holes 1-A to 8-A in the lower half of the heading were loaded first and fired together; then holes 5, 8 and 9 in the upper half and, lastly, holes 1, 2, 3, 4, 6 and 7. All holes were bottomed at 1⅝ in., 1¼-in powder being used.

The average footage was 4 ft. per round, but when the tunnel broke high, allowing the use of a 6-ft. steel for holes 3-A, 4-A, 7-A and 8-A, as much as 5 ft. could be pulled. The total time for drilling the round, which included removing the heading muck far enough back to set up the drill column, amounted to from 4 to 5 hr., the loading and shooting consuming about 3 hr. Water, when encountered, was handled by a simple steam siphon. Drill steel was sharpened by hand. Crossbits were used.

*388 Henry St., Brooklyn, N. Y.

Track was laid to within a few feet of the heading, to facilitate mucking operations. The muck was loaded into a bucket, set on a trolley, pushed by hand to the shaft, the bucket hoisted to the top and the muck dumped close beside the shaft. A 2-drum hoist was used.

The powder used was gradually reduced to 10 lb. per yd. of solid material—approximately 10 lb. per ft. of

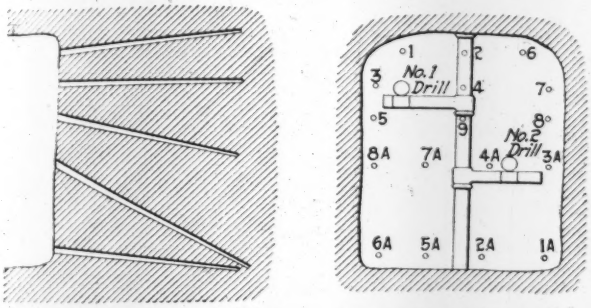



FIG. 2. LOCATION OF HOLES IN A DRILLING ROUND

advance. The rock, for the most part, broke rather small for economical handling, but this was to be expected, owing to the small size of the tunnel and the necessarily close spacing of the holes. The dynamite was 60%.

## City Bought Bridge Materials in Advance

Novel methods are being used in the City of Dayton in preparing for building three new concrete bridges—Fifth St. bridge across the Great Miami, and Webster St. and Keowee St. bridges over the Mad River. Bids are just being called for. The city manager last fall anticipated an advance in the price of steel bars and contracted for some 800 tons at a figure below $30 per ton base, f.o.b. Dayton. A contract was also placed for 53,000 bbl. of cement. Under these contracts both the cement and the reinforcing steel will be furnished by the city. Steel is now some $20 higher.

Bids are being asked by unit prices instead of by lump sum. The approximate quantities involved in the three bridges as estimated at present are given in the accompanying table. The designs were made under the general supervision of Service Director Barlow. Paul D. Fuqua, engineer in charge of bridges for the city, has direct charge of the design and construction. A local committee of architects coöperated in the architectural features. Harrington, Howard & Ash, Kansas City, were retained as consulting engineers.

APPROXIMATE QUANTITIES INVOLVED IN THREE DAYTON BRIDGE CONTRACTS

| Item | Webster | Keowee | Fifth |
|---|---|---|---|
| 1 Dry excavation, cu.yd. | 3,500 | 5,500 | 6,100 |
| 2 Wet excavation, cu.yd. | 2,100 | 2,000 | 6,100 |
| 3 Timber piles under masonry, lin.ft. | 4,700 | 7,500 | 9,500 |
| 4 Drainage system, lump sum | | | |
| 5 Porous fill over arches, cu.yd. | 3,500 | 3,000 | 3,700 |
| 6 Embankment for street approaches, cu.yd. | 6,000 | 12,000 | |
| 7 Cement, to be stored and handled, bbl. | 9,950 | 9,300 | 16,000 |
| 8 Reinforcement bars, to be handled, bent and placed, tons | 250 | 225 | 340 |
| 9 Class A concrete in spandrel and abutment walls and around ducts under sidewalk, cu.yd. | 525 | 500 | 600 |
| 10 Class A concrete in arch rings and in arch-ring anchorages in piers and abutments, cu.yd. | 2,800 | 2,300 | 3,600 |
| 11 Class B concrete in pier bases, cu.yd. | 1,010 | 1,050 | 3,100 |
| 12 Class B concrete in pier shafts and gravity-type walls, cu.yd. | 640 | 650 | 2,800 |
| 13 Class B concrete in pavement subbase, cu.yd. | 315 | 275 | 500 |
| 14 Class C concrete in abutments, cu.yd. | 2,050 | 2,250 | 2,500 |
| 15 Handrail above top of fascia, lin.ft. | 925 | 750 | 1,220 |
| 16 Cement curbs for roadway, 1:2:4, lin.ft. | 860 | 800 | 1,400 |
| 17 Cement sidewalks, 4-in., 1:2:4, sq.ft. | 7,700 | 6,100 | 11,000 |
| 18 Exposed concrete surfaces to be dressed by rubbing, sq.ft. | 11,500 | 10,000 | 10,000 |
| 19 Exposed concrete surfaces to be dressed by patent hammer or bush hammer, sq.ft. | 1,500 | 1,200 | 2,200 |
| 20 Arch rings and walls to be waterproofed, square | 270 | 200 | 450 |
| 21 Cast-iron lamp standards to be set | 16 | 16 | 28 |
| 22 Trolley poles to be set | 8 | 4 | 12 |
| 23 Electrical ducts to be laid, single duct-ft. | 19,317 | 9,800 | 29,000 |

All cement and reinforcement bars for this work will be furnished by the city. Bids for constructing these bridges will be received on or about March 15, 1916.

## Placing Warm-Air Registers in a Concrete Floor

By Arthur C. Hewitt*

The accompanying photographs show the construction work preliminary to pouring large reinforced-concrete floors in a Cincinnati theater that has warm-air registers under each seat on the first floor and on the balcony. On account of the great number of such registers some trouble in avoiding structural elements was anticipated.

In building the floors the first operation was to erect the forms and place the reinforcement. Then the center of each seat was located on the forms. There were only a few register tubes that had to be omitted because of interference with beams. After the seat centers were located, the contractor placed a galvanized-iron tube about 4 in. diameter by 10 in. long for each register. Next, the electric conduits were placed. Then the wooden sleepers

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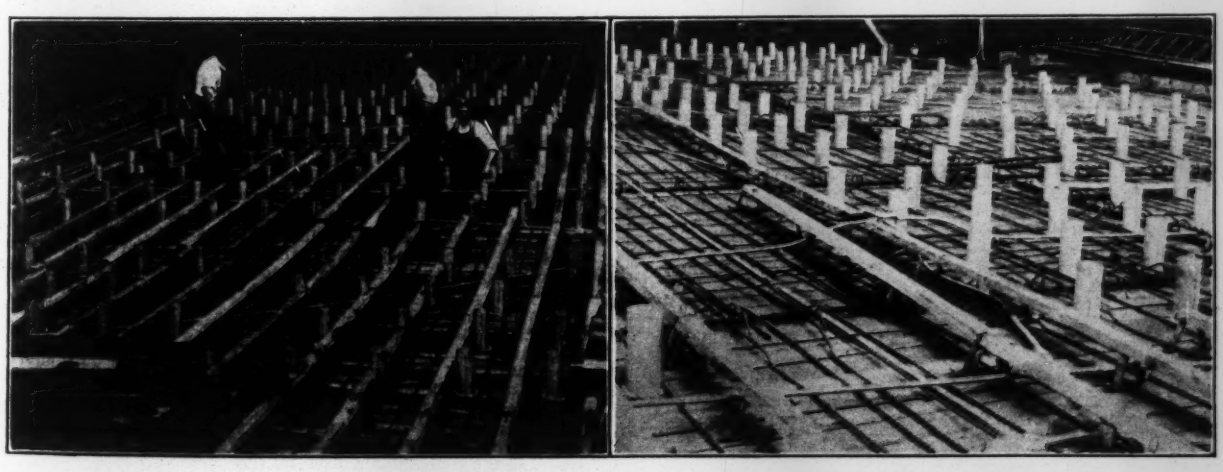

TWO VIEWS SHOWING FORMS FOR WARM-AIR REGISTERS IN REINFORCED-CONCRETE FLOOR

were set in place, supported on iron brackets made of bar iron about ⅜x1 in., bent into shape as shown in the photographs. The iron supports were left in the concrete, which was delivered in buggies, the runways for the buggies being placed on the wooden sleepers. However, before the concrete was poured, each galvanized-iron tube was carefully filled full of sand, so that no concrete would get into it. After the concrete had set and the forms were removed, there was a 4-in. hole in the floor for almost every seat. Over these holes were placed registers.

A wooden floor was built upon the sleepers set in the concrete, so that the seats could be attached to it easily. The heating plant was below the main floor and consisted, in the main, of a boiler, steam coils, an air washer and humidifier and a large electrically driven blower. The heated air was blown into a plenum chamber under the main floor, from which it was conveyed directly to the little cast-iron registers under the seats. About 25 cu.ft. per min. of heated free air is blown into the theater for each person; and as it enters from so many points the audience is not annoyed by a concentrated blast as is the case with large registers.

## Safety Scaffolds for Buildings

For the protection of painters and others working on swinging scaffolds the Illinois Factory Inspection Department and the Chicago Building Department have issued rules as to the construction of such scaffolds. Investigation showed that numerous accidents resulted from the breaking of the 2x4-in. wood bolsters of the platform or from the stirrups slipping off the bolsters. The use of wire rope is made compulsory. The essential part of the rules is as follows:

No rope of any kind is allowed for tuck pointers' scaffolds, but they must have rigid bails of ¾-in. round or square steel, forged or welded in one piece, as shown in the upper view of Fig. 1. A metal clevis fastened around the bail and one of the crosspieces of the scaffold prevents the bail from slipping over the end of the scaffold. If braces are used, they must not be nailed or screwed, but must be fastened by a hook bolt, no part of the scaffold being cut or bored in order to attach them.

The bails and stirrups of painters' scaffolds may be either of wire cable with wood putlogs, as shown in the lower view of Fig. 1, or of steel bars, as shown in the upper view. All bails, stirrups and slings must be of ⅝-in. galvanized-steel cable composed of 6 strands of 12 wires each. This large size is specified in order to allow for corrosion and careless use. Loops and stirrups for the use of bolsters must be spliced. Beneath the platform must be an additional safety cable spliced to the stirrup or bail. This will hold the platform if the bolster should break. Putlogs and bolsters must be of hickory or white oak, not less than 2 in. wide and 4 in. deep. At not less than 3 in. from each end and at the middle of the depth must be a ¾-in. bolt 6 in. long, with an iron washer on each end. Between each washer and the bolt head or nut must be a pipe sleeve 2 in. long, forming projections which prevent the bail or sling from slipping off the end of the bolster. The end of the bolt must be battered or riveted so that the nut cannot be removed. The construction of the safety bail of steel rope is shown in Fig. 2.

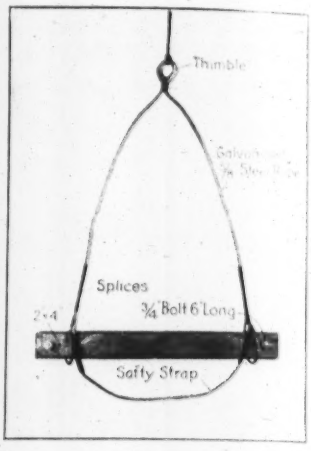


FIG. 2. SAFETY BAIL FOR SWINGING SCAFFOLDS

Some large buildings in Chicago are equipped with permanent scaffold runways on which are trolley hoists for carrying swinging platforms used by window washers, painters, tuck pointers, etc. The office building of Swift & Co. has an I-beam runway below the cornice and about 14 in. from the wall, supported on brackets. This is safer and more economical than placing and shifting roof timbers or hooks from which to suspend the scaffolds. It also prevents the nuisance due to the periodical opening of windows by the cleaners.

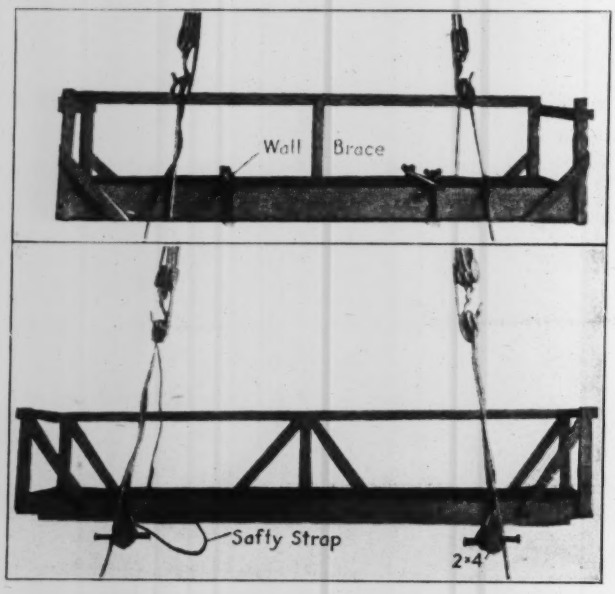


FIG. 1. SAFETY SWINGING SCAFFOLDS

## NOTES

**Teaching Lettering by Correspondence**—Something of a novelty is the announcement of a correspondence course in lettering. C. W. Reinhardt, of Arlington, N. J., for 20 years in charge of the drafting department of "Engineering News," is the author of this innovation. While it might be thought that teaching lettering, like teaching skating, is not amenable to mail-bag methods, the instruction sheets carry the spirit of the "Reinhardt system" quite remarkably.

**Street Cleaning at Utica, N. Y.,** is contracted for on the basis of great squares (10,000 sq.ft.) per day, and is done regularly during the summer and when directed in winter—about 225 days per year. The city is divided into two districts, in one of which there are daily cleaning and an hourly patrol. Here the contract price is 41c. per great square per day. In the second district there are a cleaning every two days and a patrol every 2 hr. Here the price is 26c.

**Blueprint Paper May Get Expensive**—The price of blueprint paper and blueprints is likely to advance materially in the near future. This is due to the increase in the price of all chemicals employed in the blue process and also to the big advance in paper. Red prussiate of potash has been made exclusively in Germany. Before the war it sold for 19c. per lb.; today it is practically unobtainable at $7 per lb., or an advance of 3,500%. Citric acid, from which the citrates of iron and ammonium are made, has advanced 150%; and oxalic acid, from which the oxalates of iron are made, has advanced 1,000% in the same time. The rise in the price of the cheaper grades of paper in which wood pulp is used is due to the pulp shortage caused by the scarcity of all paper-making chemicals and to the Swedish embargo prohibiting the exportation of chemical wood pulp. The better grades of paper, made of all-rag stock, have likewise advanced sharply, due also to the price of paper-making chemicals and to the shortage of supplies of rags.—New York "Commercial."

# Editorials

## Building Reinforced Concrete on the Unit System

From one point of view there is nothing new about the so-called unit system of concrete construction. It consists in general, as is well known, of casting, at a central casting yard, the individual pieces of the structure to be erected, transporting them to the place where they are to be used, hoisting them into position with some type of crane and making the final structure monolithic by overlapping the reinforcement of the units and grouting the open space between them.

It is of interest to record, however, that this system of construction appears to be coming into increasing favor among engineers. Elsewhere in this issue is described a 3-mi. trestle on the California state highway near Sacramento, just completed, in which the unit system has been applied to great advantage.

Another noteworthy use of the unit system is in the erection of railway train-sheds of the umbrella type. In the last year and a half the Southern Pacific Co. has erected sheds of this type at its station in Los Angeles and also at its station in San Francisco. The total extent of the umbrella train-shed roofs at these two terminals is about 8,000 lin.ft. At Denver the Denver Union Terminal Railway Co. has just completed about 7,000 lin.ft. of umbrella platform roof, erected on the unit system, at the Denver Union Station.

Inspection of the workmanship in these new train-sheds and in the Sacramento trestle shows results very favorable to this method of construction. Better work is secured, on the average, where concrete is made and mixed at a central yard than where it is mixed and poured on the job. The unit construction, of course, facilitates the use of the same forms over and over again, thereby greatly reducing the cost of form work. It greatly facilitates the bending and placing of reinforcing steel, since the steel required is the same for each unit and the workmen rapidly grow expert in forming and securing it properly in the molds. The thorough seasoning of the units before they are hoisted and placed in the structure helps to insure against the accidents that occasionally occur when concrete is poured in place. By a proper arrangement of the reinforcing steel it is possible to handle the units and hoist them to place in the structure more cheaply probably than the concrete materials themselves could be hoisted and poured.

In the structure referred to, moreover, it has been found possible so to arrange the reinforcement that the final structure, although built up of assembled units, is truly monolithic. It will be recognized, of course, that such structures as trestles and train-shed roofs lend themselves especially well to the unit system of construction, whereas in ordinary building construction the final monolithic result is not so easy to obtain.

It will be admitted also that where the particular appearance of the structure is a matter of importance there may often be difficulty in building by the unit system. The Sacramento trestle, for instance, could hardly be praised for its beauty, so far at least as a side view of the structure is concerned. Appearance was doubtless considered unimportant here, for the trestle is in the midst of a vast marsh, so that no one can see the trestle except travelers on the railway near-by.

Had it been necessary to consider architectural appearance in the Sacramento trestle, it would probably have been necessary to resort to pedestal piers and longer spans, which would have precluded the use of the unit system.

On the other hand, it may be pointed out that an engineering firm on the Pacific Coast has worked out a design of unit construction for three-hinged reinforced-concrete arches by which, as has been described in this journal, the half ribs are cast separately and hoisted into place by a derrick. In the design of ornamental bridge railings also it is very common to design a balustrade made up of individual units which can readily be cast from a mold that may be used over and over again; in this way a design of considerable architectural pretensions can be made at comparatively moderate cost.

*

## Concrete Roads Carpeted with Bituminous Material

A few years ago a sensation was created in street-paving circles by the announcement that a new type of permanent paving for streets and roads, which could be laid at a cost of about a dollar per square yard, had been developed in the City of Ann Arbor, Mich. The new pavement consisted of concrete covered with a thin carpet of bituminous material. Immediately the new paving was tried in many different places and with results that were almost uniformly unsatisfactory. The trouble generally experienced was the tendency of the bituminous carpet to peel off the concrete base. In Ann Arbor itself there was much trouble, but it appears from the first-hand account of Ann Arbor's experience given elsewhere in this issue that the use of this kind of pavement has by no means been given up by that city. The troubles experienced at Ann Arbor are claimed to have been due more to faulty construction than to inherent defects in this type of pavement, although it is frankly admitted that the bituminous top has required frequent repair.

This recital of Ann Arbor's experience is of especial interest, because it comes at the same time as the meeting at Chicago of the second National Concrete Road Conference, devoted to the discussion of methods of building concrete roads. If the engineers and cement manufacturers who have been active in the development of concrete roads believed that the thin carpet top for concrete roads had merit, they would long before this have undertaken its promotion, for the amount of cement required is nearly the same for this type of road as for the all-concrete type. Notwithstanding such negative

testimony there is reason to believe that the last word has not been said concerning the carpet top for concrete paving. Fifty men may fail in a given endeavor before one man supplies the single element needed for success.

The State Highway Department of California, under its chief engineer, Austin Fletcher, has scored a remarkable success in the use of a carpeted pavement, nearly a thousand miles of it having been laid. It has not, of course, demonstrated that this type of road could be made an equal success under the very different climatic and other conditions prevailing in the East. It has, however, shown that this type of road ought not to be discarded as a failure until it has been thoroughly tried in other parts of the country besides California.

The objections at once raised by advocates of the ordinary concrete road are: Why is not the concrete road without the bituminous top good enough? What is the advantage of going to the extra expense of the carpet coat on top of the concrete? It may be well therefore to state these advantages as demonstrated by the experience in California:

First, a bituminous-carpeted road is smoother and more elastic than one with a concrete top. In building a simple concrete road it is necessary to use a very rich mixture (1 : 1½ : 3 is standard practice), in order to secure a surface that will stand abrasive wear; but for roads with a bituminous top a leaner mixture may be used (1 : 2 : 4 is usual on the California state roads). The expansion joints, generally considered a necessity on concrete roads, are entirely omitted in the California roads that are to be carpeted. This makes quite a saving in cost. A further saving is made in the cost of finishing the road. No such pains and expense are necessary in finishing the road that is to have a traffic mat as on the road where the concrete itself must take the wear. Experience thus far indicates that the bituminous top is worn materially less by traffic than is a concrete surface.

It will certainly be agreed that all these things are important. Highway engineers have a constituency very difficult to please, and what is highly satisfactory in one section may awaken a storm of criticism in another. To a community that has long been used to ploughing through roads deep with the sticky mud of the Mississippi Valley, a new concrete road on which the city dweller and the farmer can run their motor cars at all seasons of the year seems like perfection itself. But build a concrete road in some sections of the East where automobile owners have become accustomed to smooth bituminous-macadam roads with an elastic rubber-like surface, and complaint is at once heard that the concrete road is too rough, that it is too hard and unyielding, that it wears the tires, that its glare is trying to the eyes.

From experience in riding over the California state roads it can be said that they have as perfect a surface as any automobilist could desire. They are so smooth, in fact, that they are habitually run over at railway speeds by motor cars. The number of cars per day is small on a large part of the mileage, but the high speed of the cars makes the service severe enough to test the quality of the surface; and very few bituminous-macadam roads would stand up as well under this fast traffic as do the California roads.

When we add to this the fact that the carpeted roads of California have been laid at an average cost of about $1.15 per sq.yd., or little if any more than the average cost of concrete roads in the East, the great advantage of this type, for California at least, appears to be demonstrated. Where frost in the ground and consequent heaving of road foundations are unknown, it is practicable to lay a concrete road only 4 in. thick; but it would be a risky experiment in New England. Even in California thicker concrete is used where adobe clay is encountered in the subgrade. The cheapness and excellence of California asphalt are incentives to the use of the bituminous top, while the high price of cement there makes the saving due to the use of a leaner mixture for the foundation concrete very important. Economizing in the thickness of the concrete is dictated not only by the cost of the cement, but by the fact that the stone has to be brought by rail in most localities.

All these, however, are minor matters compared with the one matter of making the bituminous carpet stick to the foundation concrete. Mr. Fletcher has succeeded in this on his California roads, where pretty much everyone else has failed. If this one difficulty can be overcome, there is almost certainly a big field for the carpeted concrete pavement in other parts of the country. Some claim the reason for the California success is the climate. Conditions are ideal in that climate for handling and placing bituminous mixtures, and the moderate range of temperature enables one to use an asphaltum that will not soften at the highest temperature and yet will not be brittle in the coldest weather.

Another reason for success is the practice of turning traffic over the concrete road for several weeks before the carpet coat is applied. The traffic grinds any adhering laitance or clay off the concrete, and the road is in condition to be swept perfectly clean to receive the hot asphalt. This practice has never been followed elsewhere, so far as we are aware.

It is worth mentioning, too, that the bituminous coat is applied by the state's own forces, although the concrete roads are built by contract; also, that the asphalt oil is applied hot—at a temperature of 250° to 350°—from a large tank wagon. The first coat of oil is applied at the rate of ¼ gal. to the square yard, giving a coat about $\frac{1}{4}$ in. thick. Upon this oiled surface are thrown fine crushed-stone screenings sufficient to absorb all the oil; the surplus is cleaned off, and the surface is rolled. Then a second coat of oil—¼ gal. per sq.yd.—is applied, and screenings carrying a small percentage of dust are applied. The finished coat is ⅜ to ½ in. thick and looks like an asphalt pavement.

There is reason to believe that the use of hard crushed stone instead of sand or fine gravel is one reason why the material does not roll or creep under traffic and separate from the concrete. It will be noticed also that the California carpet coat is considerably thicker than some of the carpet coats that have been unsuccessfully tried in various parts of the East.

It is fair to say that there have been a few cases where the California roads have given some trouble by peeling; also, that where the traffic is very dense, with many heavy trucks, some of the California roads have been built with the top coat 1½ in. thick. Lest it should be thought that an arid California climate is an essential to success, it may be said that a recent development is the sprinkling of the concrete before the hot oil is applied. It has been found that the oil sticks better if the con-

crete is slightly damp than it does if the concrete has even a slight coating of dust.

Mr. Fletcher states that his bituminous-carpet coat is applied to the California concrete roads at a cost of less than 1c. per sq.ft. and can be renewed, when partly worn, for less than ½c. per sq.ft. Costs would be higher than this, of course, in nearly all parts of the East; but if any such degree of success can be secured, a considerably higher cost can well be afforded.

It is well worth while for Eastern road engineers to try again on the carpet coat for concrete roads. Some of them, indeed, are already doing it. In New Jersey, for example, Col. E. A. Stevens, the State Road Commissioner, is experimenting with a wash of sulphate of zinc applied to the concrete before putting on the hot oil, and with reported excellent results in the way of firm adherence of the bituminous coat. Better construction and maintenance at Ann Arbor, more experience in California and the experimental and practical tests in other parts of the country may be expected within a few years to give data for more definite conclusions regarding bituminous-carpeted concrete roads than are yet warranted. Today it can at least be said that there is reason for being optimistic rather than pessimistic on the subject.

## City Manager's Experience with the Government of Abilene

For 18 months Abilene, Kan., enjoyed the distinction of being classed as a city administered by a city manager. The former city engineer, Kenyon Riddle, a young man from the University of Kansas, held the office of city manager under a board of three commissioners, including the mayor. In those 18 months the business of the little city was organized to run so smoothly that the commissioners decided there was no business left to require the services of a manager! So they have placed the former city manager on a *per diem* basis to do the city's engineering work.

The laws of Kansas do not specifically authorize a city-manager form of government, although the commission plan is provided for. The trouble at Abilene, therefore, was that the city manager was the creation of the commissioners; and these commissioners were salaried officials, each charged by law with the supervision of some city department. The mayor, who is chairman of the board, receives the munificent salary of $25 per month, and the other two commissioners $22 per month.

The ordinance creating the office of city manager was never legally adopted, but merely tacitly agreed to; so when the city manager commenced to shine in the administration of the city's affairs, the commissioners grew jealous of their prerogatives. The city was not big enough nor the business complex enough to require the services of all, so the city manager was the one to go.

The city manager found that the local electric company would operate the water-works pumping by electricity for one-third less than the city could operate it, so he leased it to the electric company. The city is so peaceful and righteous that the police force consists only of one day marshal and one night marshal; and the fire department consists of one piece of horse-drawn apparatus, so that the duties of the chiefs of the police and fire departments were not very onerous. There were no vice or social problems to work on, so it left little but the city engineering work to be done; and for the sake of economy, real or imaginary, the commissioners decided that this could be done cheaper on a *per diem* basis than at $1,200 a year, which was the city manager's salary.

There is a moral in this experience of Abilene, as well as an illustration of the old adage that too many cooks spoil the broth. While the experience proved that the city-manager plan is ideal for a small city like Abilene, it also proves that to be successful the office must have some foundation in law. Such powers as the city manager had were grudgingly bestowed by the commissioners, given one day and taken away the next, and whatever the city manager accomplished was only at the expense of much argument and persuasion.

## A Moribund Society Reviving

Those who went last week to Chicago to attend the obsequies of the American Concrete Institute were agreeably surprised. The victim was not dead, nor even dying. While hardly as lusty or as enthusiastic as it was in its youth, it still showed signs of a prospective rejuvenation, particularly if care is taken in the matter of diet and behavior.

To abandon metaphor, the institute is in a hopeful condition. Those in authority seem to realize that the pomp and ceremony of a major society are not part of their program and that the useful and quite necessary functions of the society can be taken care of in a much more simple manner than in the past. Freed, as it now is, from the necessarily hampering influence of solicited financial contributions, the society can proceed with such funds as its dues bring in to develop the standards of concrete construction with a single eye to the needs of the members. As these members increase, the service to them can be proportionately increased, until in time it can successfully realize the ambitious program which of late it has attempted with conspicuous failure.

In view of the necessary curtailment in effort for the coming year two definite recommendations might be made to the institute. The first is to leave out of the annual meeting all papers of mathematical complication. Such papers are distinctly not amenable to open presentation. At the same time search should be made for controversial subjects and effort made to promote discussion upon them. The second suggestion is that it is not the duty of the American Concrete Institute to prepare specifications or recommendations on subjects that are already under discussion by societies of wider scope, particularly when the institute's committee has to await the results of the larger society before taking final action. If economy of effort is necessary, here would seem to be a good place to start.

## Making Military Engineers

We noted last week the great enthusiasm with which some twenty-five hundred civil, mechanical, electrical and mining engineers in New York City started in a course of lectures aimed to help the transition of engineers from civil life into military—in the event of their need in a scheme of national defense. We note in our news columns this week the proposition of the governor of New York to build up a reserve corps of engineers.

Without in the least reflecting on the patriotism of all the engineers who are thus showing their desire to

prepare for military service we think we should caution them, and all others similarly inclined elsewhere, about the magnitude and discomfort of their task. Surely, it is well to turn aside all the easily discouraged and unfit early in the development of this movement, for the engineer troops are so vitally important that they must not be allowed to become a weak arm of the main force. We should say this even at the risk of seeming cold to any reasonable plan for the organization of our national defense—an attitude which we must immediately disclaim.

Captain Robin, in the opening lecture in New York, repeatedly harked back to the necessity of stern military training for the would-be engineer soldiers. We would fail in our patriotic duty did we not warn all office men—and the engineers behind this movement are largely tied down to their desks—that they are assuming a tedious, grueling task to get this military preparation. It means long hours of drill with arms, practice marches, camp hardships, short rations and rough fare, exposure to heat and storm. It means a program that only the physically fit should undertake.

There is no flag-waving about this sort of work—none of the glamour of parade to encourage drooping spirits. With a wise seclection of recruits the hard program will be carried through manfully, and the men will all show the true patriotism that we are sure they possess.

## When Assumptions Go Wrong

The foundation-girder problem discussed by [illegible] correspondents in this issue is a fair sample of the paradoxes that every little while rise up and bother the practicing engineer. Careful reading of the discussion and comparison with the problem as originally stated are well worth while. At the same time it is important to bear in mind that the question involved is by no means mere academic theorizing. In the particular case cited each foundation would require a ton of steel either more or less, depending on which view of the paradox prevailed. Dollars and safety are likely to be at issue in all these vexatious puzzles.

In dealing with such difficulties it will be found helpful to remember that most of them arise from a failure in the underlying assumptions—or approximations—of the reasoning employed. All engineering calculations are based on assumed conditions intended to picture the real (but variable and not definitely known) circumstances in normal cases. But in some particular application, whose appearance may not indicate that it is abnormal, the error of the standard assumptions may be very large, and the ordinary reasoning therefore may give misleading results. Then arise such paradoxical contrasts as in the foundation-girder problem.

# Letters to the Editor

## Making Military Engineers

Sir—Last week you heralded the course of military training which the engineers of New York City have enthusiastically plunged into. Now comes forward Governor Whitman with a proposal to organize a reserve corps of engineer troops out of the engineers in the employ of New York State. Undoubtedly, both these movements are highly patriotic; but I think we ought to make it very plain to everyone aching for military glory that it will be no royal road that he must travel.

My own view, from a considerable experience in state service and a pretty intimate acquaintance with a large proportion of the 1,200 engineers in the state employ, is that they undoubtedly possess the qualifications and education that would make it easier so to train them to be of value for military service along engineering lines than to train perhaps any other equal number of men that could be selected. Yet my feeling is that, until they are so trained, they would be of comparatively little value for field service.

The fact is that a large proportion of the engineers in state service have never done any work independently. They have always worked inside an office and know very little about practical mechanics or location operations. It would not be exaggerating matters to say that some of them could not saw a board squarely or drive a nail through a block without splitting it. What they need, and what they would have to have, to make them in any way efficient would be long hours and hard work under field conditions. They would be obliged to fell trees and, with absolutely unskilled labor, build temporary bridges in rapid order, dig trenches, put up entanglements and do similar work. Now, to take a group of men, of whom not one in two hundred perhaps has ever walked ten miles in a day over rough ground and who does not know the weight of a rod of barbed wire, and tell one of them to go to a certain place with a handful of men and put up a barbed-wire entanglement inside of 14 hr. would be perfectly useless.

I am not in any way deprecating engineers, but simply feel that they ought to come up to the public esteem for, and expectations of, them. I feel that they can and will do it, but they must not be expected to do it without some preliminary training that means a great deal more than simply reading books on military science, going out in khaki, living well in camp, wearing horn spectacles and looking wise.

VET.

Albany, N. Y., Feb. 18, 1916.

## Cutting a Pier in Two To Make It Stronger

Sir—The problem presented in *Engineering News* of Feb. 10, 1916, by George E. Strehan is one frequently met with in the design of footings for buildings. Unquestionably, the method shown in Fig. 2, with separate piers, would always be used if the column loads were the only loads to be considered. It frequently happens, however, in curtain-wall construction that the footings must be made continuous between the columns to carry the wall, as in the case of the corner column of a build-

ings, in which event the problem presented in Case 1 obtains. There was probably no serious contention on Mr. Strehan's part that "a part is stronger than the whole," which will hardly admit of argument in this instance.

The apparent reason for the query is the fallacious assumption that in Case 1 a uniform load is carried by

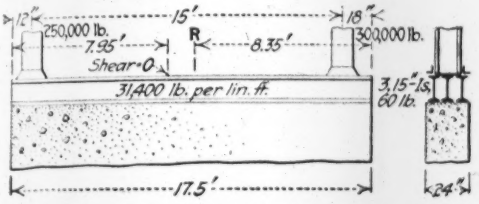


FIG. 1. FOUNDATION GIRDERS 130% OVERLOADED IF PRESSURE IS UNIFORMLY DISTRIBUTED

This and Fig. 2 reproduced from "Engineering News," Feb. 10

the beams throughout their length. This condition is impossible, on account of the deflection of the beams. In the example cited a uniform load of 31,400 lb. per ft. would cause a deflection at the center of the span of 0.24 in., figured on a simple span of 15 ft., provided the stresses were within the elastic limit of steel. It is somewhat less in the case given, on account of the overhang at the ends. The problem stated that the footings extended to rock, which permits no settlement at the ends, which would be necessary for the center to rise relatively and follow the curve that the beam would assume if uniformly loaded.

In Case 2 the load is not uniform on the piers, owing to the deflection of the beams, but it is customary to assume it so. The actual stress in the beams in this case is less than that produced by a uniform load on the pier, due to the fact that the center of pressure is not at the center of the pier. If the footings will safely carry the load, as in Case 2, why worry about getting on them a lighter load per square foot?

It is impossible to make any assumptions of elasticity of the concrete and rock foundations and construct a theoretical determination that will agree with the actual conditions. If the beams will really distribute the load in Case 1 over lengths equal to Case 2, which gives pressures within the prescribed working stresses for the concrete footing, it would seem reasonable to assume that the load acts only on this length, instead of designing them to distribute the load uniformly over their full length of bearing on the masonry, when the conditions of the problem make this an impossibility.

S. F. HOLTZMAN.

11 East 24th St., New York, Feb. 17, 1916.

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Sir—In the Feb. 10, 1916, issue of *Engineering News*, George E. Strehan discusses the case of a footing design in which two column loads, applied adjacent to the property line of a building lot, are carried by the ends of a steel distributing girder supported along its full length by a foundation wall. The intended purpose of the girder is to distribute the load over the rock foundation between the columns, but Mr. Strehan has shown that it is greatly overloaded, provided that it serves its purpose in uniformly distributing its load. However, if the center portion of the foundation wall is removed, the girder is of sufficient strength.

That the assumption of a uniformly distributed loading, with a consequent overstressed condition of the steel, is erroneous is shown by the following: A uniformly distributed load of 550,000 lb. applied to the beam arrangement, shown in Fig. 1, would cause a deflection of approximately 0.75 in.

Assume a foundation arrangement similar to that of Fig. 2, with the steel distributing girder removed and with a foundation pier 2 ft. square directly under the column load of 250,000 lb. The load on this pier would be 435 lb. per sq.in., and the total deformation or shortening due to this load, assuming a modulus of elasticity of 2,500,000 and the deformation to occur in a length of 60 in., would be approximately 0.01 in.

While the pier of comparatively small cross-section would be deformed only 0.01 in., the addition of the distributing girder and the solid foundation wall would apparently result in a further deformation of 0.74 in., the intensity of loading being at the same time reduced from 435 to 109 lb. per sq.in.

It may be concluded from these results that the distributing girder is not overstressed, that the foundation

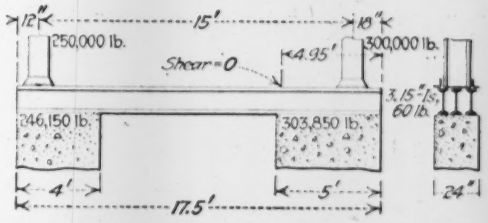



FIG. 2. SAME GIRDERS MADE SAFE BY CUTTING OUT MIDDLE OF PIER

as shown in Fig. 1 is not unsafe and that the arrangement of girder and continuous foundation wall does not result in a uniform distribution of load. In this particular case the girder receives only a small part of the load that it is capable of carrying.

G. M. WILLIAMS,
Bureau of Standards.

Washington, D. C., Feb. 15, 1916.

## Governmental Publication of Railway Statistics

Sir—On page 116 of *Engineering News*, Jan. 20, 1916, I notice some comment on the twenty-seventh annual report of statistics of railways. Your principal criticism is directed to the lateness of the date at which this report is issued. I quite agree with you that it is desirable to have information of this character published at as early a date as practicable, but the commission's publication of railway statistics is not confined to this report.

The commission receives monthly reports of railway earnings and expenses, which the companies are required to file with us within 30 days after the close of the month. We feel that we are getting the figures together and giving them to the press with all the promptness which can reasonably be expected, when we supply them within 40 days after the close of the month covered by the reports.

Under the statute the carriers have three months in which to file their annual reports of operation, and frequently it is necessary for them to secure additional time,

so that it is often the end of October before the reports of the larger carriers can be filed with the commission.

In order to give publicity promptly to these figures the division of statistics of the commission compiles and publishes each year what is known as the "Preliminary Abstract of Statistics of Common Carriers." This abstract is based upon the figures as reported, with such minor corrections as can be made without correspondence with the carriers, and we are usually able to send the copy for this compilation to the Government printing office early in December. Owing to the great pressure of work upon the Government printing office at this season of the year our work has to be subordinated to certain other printing, and it thus happens that this abstract cannot usually be published before some time in January, but we are generally able to get it into the hands of the public within seven months after the close of the year to which it relates, or within three months after most of the reports from which the figures are compiled have been filed with the commission.

Our annual volume, noticed in your issue of the 20th instant, is a much more elaborate report and necessarily takes a much longer period of time to prepare. If the figures as submitted by the carriers were in all instances correct, it could be published at a much earlier date, but unfortunately the reports of the carriers almost invariably contain discrepancies the detection of which requires a very careful examination in the office of the division of statistics and correspondence for the correction of errors and discrepancies thus discovered. This examination and the correction of the discrepancies require a considerable period of time, owing to the great number of reports that we have to handle. This largely causes the delay complained of. We have always thought it advisable, however, to put out in permanent form only such figures as have been carefully tested in order that they may be as accurate as practicable.

For the latest annual reports resort may be had to the preliminary abstract of statistics or to the reports themselves, which are public records, and for the current year's figures fairly satisfactory information regarding earnings and expenses may be derived from the monthly reports.

W. J. MEYERS,
Washington, D. C., Jan. 26, 1916. Statistician.

**The Increase in Automobile Registrations** in the State of Pennsylvania between 1914 and 1915 was nearly 44,000, or more cars than there were in the state in 1911. The receipts of the State Highway Department from the registration of 154,697 cars, 16,907 motor cycles, 2,297 tractors and trailers and from dealers' and chauffeur's licenses amounted to $1,631,169 during the current year. These numbers do not include the free registration of fire, police and ambulance motor vehicles.

**Sewage Disposal for Los Angeles County, California,** is to be studied by a commission of sanitary engineers appointed by the County Board of Supervisors on Jan. 24. This action is the outcome of opposition to the proposed joint sewage-disposal works for Pasadena, South Pasadena and Alhambra, for which 600 acres was bought some weeks ago. The commission consists of Prof. Charles G. Hyde, University of California, who was named chairman; Thomas A. Gillespie, sanitary engineer of the State Board of Health; George P. Robinson, of the city engineer's office, Los Angeles; A. H. Koepig and A. M. Brosius. The commission has been instructed to study various methods of treatment in their relation to odors, flies and mosquitoes, and danger of the spread of disease germs; whether sewage effluent could be safely used for irrigation; the advisability of an ocean outfall and the depth and the distance from the shore at which treated sewage should be discharged.

# Rogers Pass Tunnel Method Chosen for Economy

By J. G. SULLIVAN*

Under the heading "Two Tunnels to Make One," an editorial in *Engineering News* of Nov. 18, 1915, p. 993, discussed the method of driving used in the Rogers Pass tunnel of the Canadian Pacific Ry. The criticisms and questions of the editorial would appear logical and reasonable if speed were the only object in mind. But the most important factor—and the one that must govern all good engineering—is the factor of economy, and this alone was considered in making our decision. Time was a factor in this larger question of economy, but much smaller than some others. We considered time to be worth approximately $750 per day.

## The European Method Barred by High Cost of Labor

In a circular letter sent out to contractors inviting them to bid, dated April 8, 1913, the following statements appeared:

> The necessity for this tunnel is so great and the expenditure so large that it would be worth considerable money to this company to have the tunnel completed as soon as possible. Therefore, everything else being equal, the party who will guarantee completion in the shortest time will be the party who will receive the work.
>
> I would be glad if you would give us prices on the European method of tunneling, which is to drive a very small heading and take out the bench, working from several headings into this small drift. Tunnels in Europe have been driven by this method at two or three times the speed that any tunnel was ever driven in the United States or Canada, and I would like to be able to place before the management figures for doing this work according to this method. I would be glad if you would state in your proposal the amount per day that you would be willing to have inserted in a contract to be paid as a bonus for time saved over the agreed time, the same amount to be exacted as a penalty for the time lost, being the time between the fixed day of completion and the actual date of completion. We are of the opinion that this sum should be about $750 per day.

The writer, however, was aware at this time that the chances of American contractors tendering a reasonable figure on the European method were very remote; in fact, after having studied the prices paid for labor in some of the large Swiss and Italian tunnels, we were forced to the conclusion that theirs was not a practicable method for this country, where labor is so expensive. On Mar. 13, 1913, the writer reported to the management his ideas on this subject. To quote from that report:

> Referring to the progress that we hope to make in the driving of the Rogers Pass tunnel. I advised you in my report of Oct. 22 regarding the relative speeds of driving tunnels on the American continent compared with those that have been driven through the Alps. I have given the matter considerable study since and have come to the conclusion that the European method of driving a small lower heading and stoping out the remainder of the tunnel is too expensive on this side on account of the difference in the cost of labor.

I then described in general the methods which we proposed and which we have practically followed out.

## Economy Required a Steam-Shovel Method

The method desired was one by which we would be able to handle the minimum amount of material by hand and the maximum amount by steam shovel. By driving a heading in the center of the excavation required for the completed tunnel, making this heading just large enough

*Chief Engineer, Canadian Pacific Ry., Winnipeg, Man.

so that we could handle steel that would reach from the perimeter of this advance heading to the perimeter of the completed tunnel, and then drilling holes at right angles to the axis of the tunnel we would be able to shoot at once the entire section required for the enlargement of the tunnel.

This shooting was done in the following manner: Six holes at the bottom of the tunnel having been previously shot out, six holes would be loaded in the next ring back in the bottom and all the side holes of the first ring up to a point a little over halfway up the sides of the tunnel. This method would continue until the tunnel began to fill up, and then some of the advance rings in the upper portion of the tunnel would be shot and this process continued until the tunnel was choked full of muck.

There were usually shot about six rounds (or 30 ft.) of holes before the steam shovel started to clean up. In some of the harder rocks that shot well it was possible to shoot a greater distance. The most shooting done at one time up to date is 84 ft. This method enabled us to handle about 85% of the total excavation of the tunnel proper with steam shovels.

This result was the prime object to be obtained, and how to accomplish it without interfering with the work of the shovel was solved by driving the "pioneer heading," or tunnel, to be used as a bypass for removing the muck from pioneer and advance headings, for ventilating ducts, air, water-pressure pipes, etc. The result was that work was carried on continuously at all points irrespective of shooting at other points, work only stopping at the local point at which shooting was being done. As there were no air pipes or water pipes or any obstruction whatever passing over the muck pile in front of the shovel, the shovel was able to do 85% of the total excavation almost as easily as if it had been an open cut.

### Excavation Done at Remarkably Low Cost

Our expectations have been more than realized, as is proved by the speed and the cost of the work. The cost of driving this tunnel through rock, including in this price the cost of driving 19,610 lin.ft. of pioneer tunnel, 12 crosscuts each about 40 ft. long, installation of plant (including freight), the proportionate cost of building about 5 mi. of temporary railway tracks, and other overhead charges, plus 10% on all expenditures, will amount to a little less than $5 per cu.yd. for excavation in the tunnel proper.

I may add further that, in reply to my invitation of April 8, 1913, the contractors who are doing the work, having in mind the method which was later adopted and which was suggested by myself and one of their engineers, bid $6.10 per cu.yd., with a time limit of 42 months from date of signing contract. Other responsible and supposedly expert tunnel contractors bid from $8 to $11.25 per cu.yd., with time limits varying from 42 to 48 months. I do not know what method these latter contractors proposed to employ, but I always presumed it would be some modification of the European method.

We also received an estimate of $5.50 per cu.yd. for tunnel excavation from a reputable American expert tunnel-contracting firm which would do the work on a percentage basis and would be willing to forfeit a large portion of its percentage if it did not keep the price within the estimate. However, it estimated the time at 58 months. At the speed this firm promised for driving tunnel in rock and figuring the time it took our contractors to get into rock at both ends on account of heavy cuts, it would have taken 8 or 9 months longer than the estimate, or something over 5½ yr. The method that it proposed to employ was to remove tunnel right from a face, loading muck by steam shovel.

### Shovels Advance More Than Three Miles per Year

Now as far as speed is concerned: On account of large cuts and soft ground at the approaches, the shovel doing the enlargement work did not reach the rock on the east end until Jan. 1, 1915, and on the west end until Feb. 1, 1915. On Dec. 19, 1915, when the heading was completed, the shovels were 2 mi. apart. Had the conditions been such that we could have got into the tunnel proper at earlier dates, I do not think there is any European record that would not have been broken. We have been driving tunnel at the rate of better than 3 mi. per yr. for the last year. During the month of January, 1916, the shovel in the east end made an advance of 946 lin.ft.

In conclusion I wish to say that in Europe, where drill runners, as I understand, get 90c. to $1.25 per day and laborers something like 75c., the method followed in driving the Loetschberg tunnel may be economical as well as rapid; but all the evidence we have goes to show that where we have to pay from $4 to $5.50 per day for drill runners and $2.25 to $3 for laborers the method we have adopted is by far the most economical.

## San Luis Rey River Floods of January, 1916

By Edward R. Bowen*

Marooned at Oceanside during recent floods in southern California, the writer embraced the rare opportunity thus afforded to study the January flood conditions of the San Luis Rey River. The results of the study are here given.

The San Luis Rey River flows in a general westerly direction for about 60 mi. and drains an area of 564 sq. mi. above its mouth at Oceanside. The river valley is quite level and has the same slope as the river. The confining rock at various points along the valley opens out and again closes in, thus forming a series of basins. The stream usually flows as a surface stream throughout its length, from a few days to a few months in the spring of each year.

The rainfall, while light along the seaboard, is considerable in the high mountain regions. San Diego, with a 65-yr. record, has a mean of 9.61 in. Oceanside has a 23-yr. mean of 11.15 in., while Nellie, at El. 5,350 ft., has an 11-yr. mean of 48.57 in. At the Warner dam site, El. 2,700, the mean rainfall for the past 4 yr. has been 30.34 in. The accompanying table shows the daily precipitation during the month of January at all the stations in and around the drainage area from which rainfall records are available at the present time. During the season 1915-16 the rainfall at Oceanside up to Jan. 1 was 4.06 in.—0.61 in. during November and 3.41 in. in the early part of December, so that the ground was in a moist condition at the beginning of January when the storms commenced.

*Engineering offices of J. B. Lippincott, Central Building, Los Angeles, Calif.

The recent floods occurred on Jan. 17 and 27. The precipitation up to Jan. 16 was generally a fine gentle rain, most of which was absorbed by the soil. The rains of Jan. 16 and 17 fell upon the water-soaked ground and appeared almost entirely as runoff. The flow of the river at its mouth on Jan. 15 was about 200 sec.-ft. No measurements were available for the next day. The river was in flood on Jan. 17, the crest reaching the mouth of the stream about noon of that date.

FIG. 1. GENERAL VIEW, SHOWING SAN LUIS REY VALLEY DURING FLOOD OF JAN. 27, 1916

### Gaging First Flood at Oceanside Bridge

An estimate of the flood was made by taking soundings across the river from the highway bridge at Oceanside. Current-meter measurements were made for a distance of 280 ft. out from the south bank, at which point the current was so swift that it was impossible to keep the meter in the stream. Floats were then timed both in the part of the stream over which current-meter measurements had been made and in the main current. The velocity in the main body of the stream was computed as 16 ft. per sec. and the runoff at 27,000 sec.-ft. The river at that time was 530 ft. wide, and its greatest depth was 14.5 ft.

Quantities of drift piled up against the Santa Fe trestle bridge, which crosses the river about 2,000 ft. west of the highway bridge at Oceanside, and finally washed out five bents. The San Luis Rey bridge, 4 mi. upstream, had about 100 ft. of its approach washed away. The

FIG. 2. WRECK OF PUMPING PLANT

Bonsall bridge, 17 mi. upstream, a concrete and timber structure, was washed out.

The river spread over large areas of bottom and bench lands, destroying crops and in many places depositing large quantities of débris and silt. No lives were lost. All telegraph and telephone communication was cut off, and transportation by highway was interrupted for three days, while railway communication was cut off indefinitely. The roads up the river were passable only on horseback, and then with difficulty.

The river fell rapidly after 6 p.m., Jan. 17. On Jan. 18 an estimate was made in a manner similar to that previously described, except that the floats were timed and the station at which they were measured was noted. These surface velocities were then plotted as ordinates against the stations across the stream as abscissas, and intermediate velocities were taken from the curve. The velocity curve thus obtained was very consistent in its form. The result of this estimate showed 16,000 sec.-ft. in the morning and about 18,600 sec.-ft. in the afternon. Jan. 19 the river flowed 10,300 sec.-ft. in the morning and 10,200 in the afternoon. Further measurements were not available until Jan. 26, when the river flowed 1,757 sec.-ft. by current-meter measurements from the Oceanside bridge.

### Stream-Flow Measurements for Second Flood

On the afternoon of Jan. 26 the second division of the storm arrived at Oceanside. From this time up to 8 a.m., Jan. 28, the rainfall at Oceanside amounted to 3.62 in. The rainfall in the mountainous districts (see table) was substantially greater. This storm produced a flood of much greater volume than the preceding one.

A measurement of the river at the Oceanside bridge about 10 a.m., Jan. 28, showed 9,600 sec.-ft. An esti-

INCHES OF RAINFALL IN AND NEAR SAN LUIS REY RIVER DRAINAGE BASIN, JANUARY, 1916

| Date, Jan. | San Diego, El. 93 | Oceanside, El. 67 | Escondido, El. 657 | Warner's Dam, El. 2,700 |
|---|---|---|---|---|
| 1 | 0 | 0.26 | 0 | 0 |
| 2 | 0 | 0 | 0 | 0.23 |
| 3 | 0 | 0 | T* | 0 |
| 4 | 0 | 0.10 | 0 | 0 |
| 5 | 0.01 | 0.02 | 0.07 | 0.16 |
| 6 | 0 | 0 | 0.27 | 0.41 |
| 7 | 0 | 0 | 0 | 0 |
| 8 | 0 | 0.26 | 0 | 0.44 |
| 9 | 0.20 | 0 | 0.21 | 0.21 |
| 10 | 0.27 | 0.33 | 0.05 | 0 |
| 11 | 0 | 0 | 0.62 | 1.78 |
| 12 | 0 | 0 | 0 | 0 |
| 13 | 0 | 0 | 0 | 0 |
| 14 | 0.53 | 0.55 | 0 | 0 |
| 15 | 0.09 | 0.20 | 0.68 | 1.13 |
| 16 | 0.95 | 1.80 | 0.40 | ... |
| 17 | 1.55 | 1.23 | 3.30 | 6.37 |
| 18 | 0.31 | 1.20 | ... | ... |
| 19 | 0.80 | 0.26 | ... | ... |
| 20 | 0 | 0 | ... | ... |
| 21 | 0 | 0 | 6.49 | ... |
| 22 | 0 | 0 | ... | ... |
| 23 | 0 | 0 | ... | ... |
| 24 | T* | 0.54 | ... | 0.22 |
| 25 | 0.21 | 0.08 | 0.21 | 0.50 |
| 26 | 0.22 | 2.02 | 0.61 | 2.70 |
| 27 | 2.19 | 1.60 | 1.71 | 7.73 |
| 28 | 0.06 | 0.06 | 4.13 | 0.04 |
| 29 | 0.17 | 0.31 | 0 | 1.01 |
| 30 | 0 | 0 | 0.45 | 0 |
| 31 | 0 | 0 | 0 | 0 |
| Total for month | 7.56 | 10.82 | 19.20 | 22.93 |
| Average for season | 9.69 | 11.15 | 15.51 | 30.34 |

*T = Trace.

made at 5 p.m. showed 55,400 sec.-ft. Float velocities of 25 and 30 ft. per sec. were frequently observed. These velocities were so great that soundings could not be obtained all the way across the river, it being impossible to get the weight to the bottom.

Sections of the Santa Fe trestle went out at intervals during the afternoon. The highway bridge at Oceanside was a reinforced-concrete arch bridge of six spans of about 100 ft. each, built by the unit system of construction, with three-hinged arches. This bridge seemed to be withstanding the flood, although there were some vibration and great scouring around the piers. The center pier showed some cracks on the downstream side. The river continued to rise, reaching its crest about 9:30 p.m., at an elevation of 2 ft. above that at the time the last-noted measurement was made. The remaining Santa Fe trestles had already been swept away. About this time the concrete bridge at Oceanside went out. It is impossible to state exactly how the failure occurred, it being too dark to see; but the writer believes that the center pier was undercut and washed out and that the rest of the bridge, owing to its construction, fell like a pack of cards.

Soundings made of the river showed deep depressions around each of the piers, and the scour at these points during the flood was very great. The peak of the flood was probably in excess of 70,000 sec.-ft. The entire San Luis Rey Valley was inundated (Fig. 1), the stream extending from hill to hill—a distance of probably 1½ mi.—and covering an area of over 1,000 acres. The drift along the county road on the south side of the valley indicated a 6-ft. depth of water at that point.

### Widespread and Varied Damage Caused

This valley had been used for beet culture and alfalfa. A large pumping plant erected by the South Coast Land Co. was completely lost (see Fig. 2). The pump house was a reinforced-concrete structure, and the equipment was of the highest type of modern water-works steam-driven pumps, completed about a year ago. It was located about a mile from the channel of the river, and flood-protection work of levees and willow planting had been built along the bank of the stream for a half-mile above the plant. It is probable that the enormous quantities of silt and débris carried by the stream in the first flood, together with that carried by the subsequent one, so built up and raised the old channel of the river that it was at a greater elevation than the adjacent valley lands. When the second flood came down this channel, its banks were overtopped and a new channel formed, cutting across the valley in a northeast to southwest direction, taking a long sweep to the westward at about the middle of the valley and returning to an old course near the narrows. The pumping plant was directly in the course of the new channel of the river, and the foundations of the building and pumps were undermined. All that remain visible of the plant are the domes of the water end of the pumps. It is probable that the main pumping engine may be salvaged.

The municipal pumping plant of the City of Oceanside is located in the narrows of the stream; and although the water rose to a depth of 2 ft. over the engine-room floor, the direct current of the stream did not reach the pumping plant, and no material damage occurred. It was out of commission for a couple of days, and temporary pumps had to be installed to maintain the domestic supply for the city.

All communication to and from Oceanside was interrupted by the second flood, and the city was isolated for a period of five days. All farms in the valleys of the lower river were completely destroyed and three people were drowned. The entire valley is covered with deposit of sand and silt to an average depth of at least 3 ft.—and in many places as much as 6 ft. Conditions along the upper river are not so bad. The valleys are more constricted and the stream better confined, although all crops along the bottom lands have been ruined. All the stream-flow gages and cable stations along the river have been washed out.

## Concrete-Roads the Subject of National Conference

Two years ago about 200 engineers interested in the construction of concrete roads and streets met at Chicago to discuss methods of securing the best results in this work. A second national conference on concrete-road construction was held in Chicago from Tuesday to Friday of last week, with a registered attendance of about 600.

How rapidly the concrete road is advancing in public favor is shown by statistics collected by the Association of Portland Cement Manufacturers. The total of concrete roads and streets laid previous to 1915 was about 1,600 mi. of roads and 19,000,000 sq.yd. of paving. During 1915 about 1,000 mi. of concrete road was built, and the total area of concrete street pavement laid was about 5,500,005 sq.yd.

There is every indication that the skepticism of many engineers and of the general public respecting concrete roads is rapidly passing away and that the great advantages of the properly built concrete road in durability and low cost of repair are beginning to be understood.

The chairman of the conference was Dr. W. F. M. Goss, Dean of the Engineering Schools of the University of Illinois, and the secretary was J. P. Beck, Secretary of the Association of Portland Cement Manufacturers. The conference held six sessions, at which were presented the reports of 17 different committees, each dealing with some special aspect of concrete-road work. Several papers and addresses by individuals were also presented, and most of the committee reports received extended discussion.

A committee on resolutions was appointed at the opening of the conference, made up of about a score of the engineers of most experience in concrete-road work. To this committee all the committee reports and discussions were referred, and it brought in a report at the closing session summarizing the reports as a general statement of good practice in concrete-road work. This report after some amendment was adopted by the conference.

The entire proceedings of the conference are to be edited and published and, as was remarked in the sessions, will constitute the bible of the concrete-road industry. Space permits only a brief review at this time of some of the salient facts brought forward during the conference.

**Cost of Concrete Roads**—The Committee on Cost of Constructing Concrete Roads, through its chairman, C. J. Bennett, State Highway Commissioner of Connecticut, reported that the cost per square yard of the concrete roads built in 1915 varied from 95c. to $1.28. In New

York and Ohio, in each of which states a large mileage of concrete roads was built in 1915, the average cost was 98c. and $1.02 respectively.

Cost of Annual Maintenance—Much skepticism has prevailed in the past concerning the reported low figures of maintenance cost on concrete roads. The chairman of the committee on this subject was A. H. Hinkle, Deputy Commissioner of the Ohio State Highway Department, which state has about 260 mi. of concrete roads that have been laid 1 to 5 yr. The actual figures of maintenance cost on 127 mi. of these roads for work on the concrete surface alone, including cost of superintendence and including also the entire cost of a number of tar kettles purchased during the year and still in use, was $29 per mi. Cost data of maintaining concrete state roads in Illinois show figures varying from 0.22c. per sq.yd. per annum to $0.32c., according to the kind of equipment used. This is equivalent to a cost per mile of a 15-ft. wide road of $19.36 and $28.16 per annum. In view of these and other figures the committee reported that "a conservative estimate for concrete roads 1 to 5 yr. old is a maintenance cost of less than ½c. per sq.yd. per yr." This is equivalent to less than $44 per mi. of 15-ft. road.

Details of Construction—In the reports of the committees reviewed, as well as in all the proceedings of the conference, it was urged that high quality of materials and workmanship is essential to the success of concrete roads. Twelve essentials of good practice were summarized as follows in the report of the Committee on Resolutions:

1. Drainage of the road bed is vital.
2. Subgrade must be of uniform density and should be compacted.
3. Aggregates must be clean and hard.
4. Fine aggregates (sand) should be coarse and well graded.
5. A rich mixture must be used.
6. Materials must be accurately proportioned.
7. Mixing must be most thorough.
8. Sloppy concrete must not be used
9. In general the use of reinforcement is justifiable.
10. Inspection must be intelligent and thorough.
11. Concrete must be protected from rapid drying.
12. Pavements must not be opened to traffic too soon.

Probably the most common fault in present concrete-road building is mixing the concrete too wet. In many of the committee reports it was urged that a stiff concrete be used and not a soft, mushy mass. The conference agreed after much discussion that a width of 10 ft. was desirable for single-track roads and 18 ft. for double-track roads, to give greater safety for high-speed traffic. On the other hand, many roads of 8-ft. and 16-ft. widths are being built. Stone or gravel shoulders to increase the width of narrow roads were not favored, as the high cost of maintaining them, added to their first cost, makes their total expense greater than that of an equal width of concrete that costs very little to maintain. A minimum thickness of 6 in. was recommended and a crown of $^1/_{50}$ to $^1/_{100}$ of the width.

Proposals to recommend the use of reinforcement in concrete roads were strenuously opposed in view of the fact that only a small percentage of concrete roads thus far built are reinforced. The specifications adopted, however, advise the use of transverse reinforcement on roads over 20 ft. in width, to prevent longitudinal cracking. A spacing of about 50 ft. was recommended for transverse joints. The tendency of practice is away from the use of metal joint plates.

By a resolution adopted at the closing session the conference declared that a third conference should be held and continued the advisory committee with power to fix the date and make necessary arrangements for such a conference. Copies of the proceedings of the conference just closed, when published, may be obtained from the secretatry, J. P. Beck, 208 So. La Salle St., Chicago.

## American Concrete Institute Is Much Alive

Contrary to the previously held impression, the twelfth annual convention of the American Concrete Institute at Chicago, Feb. 14 to 17, did not prove the dying effort of the organization. Over 450 members and guests were registered, the nine sessions were well attended, and the board of direction was so impressed with the spirit of appreciation of the association held by those present that a proposed dissolution scheme was not only abandoned, but the program for the ensuing year was laid out with a renewed confidence, somewhat absent from the deliberations of the board during the last few years.

On account of the withdrawal of certain large contributions from the cement and allied interests the financial prospects of the coming year are not so bright. To meet the new conditions, the society is to enter into a period of rigid economy, which imposes the giving up of a paid secretary and an office, as well as a reduction in the volume of printed proceedings. At the same time Leonard C. Wason, of Boston, who was reëlected president, has offered gratis the use of his office force to conduct the necessary headquarters affairs; Walter S. Hinds, of the Atlas Portland Cement Co., New York, will serve as Secretary without compensation and will take care of the editing and the publication of the proceedings. It is hoped in the future to devote more attention to committees, so as to continue at a maximum efficiency their very valuable contributions to the knowledge of concrete.

The program of the convention was overloaded, in which respect it did not differ from the programs of previous years or, in fact, of most technical meetings. The president very wisely had set a time limit on papers and discussions; but unfortunately a strict adherence to the rule acted to cut short useful discussions, in order to admit useless papers that were on the program and therefore had the right of way. Had the pure descriptions of structures and the mathematical discussions been left from the program, more time would have been had for the practical and controversial debate that, after all, is the only matter which can be appreciated or understood on the floor of a convention.

### Committee Reports of the Institute

No committee report presented contained a specification or recommended practice that had had the required publication before the meeting, so no final action on committee reports was taken, except that the definitions submitted last year by the Committee on Nomenclature were adopted and, in a slightly altered form, sent to letter ballot. In brief, the action of the committees was as follows:

Insurance—Progress report and suggestions as to methods of collecting data. Building Blocks and Cement Products—Combination of specifications on blocks, bricks and stone into a new suggested specification and a new

specification on drain tile; ordered printed. Reinforced Concrete and Building Laws—Report on proposed revised methods and suggestions as to building work; referred back to committee. Fireproofing—Advised waiting for results of proposed tests on columns under load and fire conditions soon to be conducted at Underwriters' Laboratories. Treatment of Concrete Surfaces—Slight changes in specification and report that Bureau of Standards is conducting service tests on stucco. Sidewalks and Floors—Proposed program for future work. Concrete Roads and Pavements—Minor changes in specifications and elimination of any specific description of reinforcing methods. Concrete Aggregates—Progress report. Reinforced-Concrete Highway Bridges and Culverts—Extensive and valuable report on methods of design and construction.

### Flow of Concrete Under Early Loads

The most interesting group of papers was the one on tests, in which a generally unrecognized phenomenon was emphasized. The matter may well be abstracted in the following paragraphs from a paper by A. T. Goldbeck and E. B. Smith on "Tests of Large Reinforced-Concrete Slabs," the slabs being those recently tested by the Office of Public Roads at Washington:

> One of the important facts which these tests brought out, and which this paper seeks to impress upon concrete engineers, is that the time element is a very large and important factor in determining stress values and their distribution from the fiber deformations. Nearly all materials of construction with which the engineer has to deal will yield an approximately constant relation, independent of the time, between the load and its accompanying deformation within the limit of elasticity. This is not true in the case of concrete, which exhibits a marked flow, or molecular adjustment, under working stresses extending over time periods of several weeks. During the tests of one of these slabs in August, 1914, this phenomenon was first discovered, and a series of experiments was begun to obtain data regarding the flow of concrete. Since it is not the object of this paper to discuss the facts or the theory of this phenomenon, the following statement will be sufficient for the present purpose:
>
> When concrete is subjected to a comprehensive fiber stress of 700 lb., the immediate fiber deformation is only about 50% of what it will be if the stress is maintained for three weeks. Within 24 hr. after the application of the load the deformation has increased an additional 20%; during the first hour the deformation may show a change of 5% or more. Furthermore, the recovery of the deformations, after the removal of the load, is slow and not complete.
>
> This fact is mentioned here to show the importance of taking all the deformation readings at approximately the same time after the load is applied, and in each case a new set of deformation readings under more load must be taken just before each load application.
>
> The authors' purpose is accomplished by emphasizing that such tests, and in fact all tests for stress and deformation values in concrete, must be conducted only with a full realization of the importance of the time factor. The immediate fiber deformation is the value which should be used, and not the value obtained after a long-time suspension of the load, nor that which contains the effect of several applications and removals of loads.

This condition was emphasized in a paper by Prof. A. H. Fuller and Prof. C. C. More, University of Washington, and in the discussion of the paper by Professor McMillan, of the University of Minnesota. The tests reported show that, for a considerable time after initial setting (in some cases now nearly two years), deformation in the concrete continues. In one 10x10-ft. slab under test by Professor McMillan the deflection is constantly increasing under the designed load of 100 lb. per sq.in., but at the same time the concrete shrinkage is so great that the steel in the lower part of the slab is in compression. None of the speakers would offer any comment on the specific reference of such observations to the future integrity of concrete structures.

Prof. Duff A. Abrams, Lewis Institute, Chicago, in his paper "Tests of 885 Sands," presented some exceptionally complete analyses of tests made on sands from widely distributed localities and of varying composition. Professor Abrams has established a new constant, called the "fineness modulus," which is obtained by adding together the percentages retained on the Fuller-curve screens, and has discovered what he considers to be a comparable relation between the behavior of a sand as a fine aggregate and that modulus. His tests are on a great many specimens and cover a wide variety of conditions.

In discussion Rudolph J. Wig, Bureau of Standards, repeated what had been stated several times before on the floor, that the gradation of a sand was not an indication of its value as a concrete aggregate. He claimed that Professor Abrams' results were mostly on grand averages and that the individual sand test varied so from the average of its class as to make that average valueless. Mr. Wig's deductions are based mainly on an extensive sand-test series to be published by the Bureau of Standards.

### Design and Construction in Concrete

Flat-slab floors were treated in a remarkably clear and impartial paper by Alfred E. Lindau, Corrugated Bar Co., Chicago, entitled "Construction of Flat-Slab Floors." In it all the various systems were described. Other papers on the same subject were by Edward Smulski, Boston, Mass., who described a flat spiral system of reinforcement of his own, and by Prof. H. C. Eddy, Minneapolis, who elaborated his now well-known, though not so well understood, theory of the action of flat plates. Professor Talbot, University of Illinois, pointed out that Professor Eddy reduced the bending moment in a slab by one-half by assuming that a crossed system of embedded rods acts as a continuous metal plate and by another half because of the tensile resistance of the embedding concrete. He thus assumes that the steel in a flat slab is subject to only one-fourth of the stresses as calculated by the beam theory. Professor Talbot said that such a statement is so absurd that, coming from so eminent an authority on mathematics as Professor Eddy, it could hardly be taken seriously and must be assumed to be intended as a joke or a hoax on the profession.

Other papers on design and construction, best explained by their titles, were: "Unit Costs in Construction," by Sanford E. Thompson, Newton Highlands, Mass.; "Design of Concrete Buildings," W. P. Anderson, Ferro-Concrete Construction Co., Cincinnati, Ohio; "Design of Reinforced-Concrete Foundations for Buildings," R. L. Bertin, New York City; "Forms for Concrete Work," R. A. Sherwin, Aberthaw Construction Co., New Haven, Conn., and "Reinforced Concrete in Sewer Design," W. W. Horner, St. Louis, Mo. The paper on concrete chuting, by C. C. Brown, Indianapolis, Ind., was a statement of the fundamental principles involved in such work, but studiously avoided any reference to the absorbing question of the final strength of concrete deposited in such manner. There was no discussion.

A number of papers on completed concrete structures were included in the program, as well as a most interesting résumé by Prof. W. K. Hatt, Purdue University, of the history of reinforced concrete, as indicated by the literature of the subject prior to 1900.

# News of the Engineering World

## Engineer Wanted on Public Service Commission

House Bill No. 28, now in the Committee on Corporations of the Maryland legislature, proposes to amend the public-service law of the state by adding at the end of the third paragraph of Section 414 the following, "and one of said commission shall be an engineer." The Engineers Club of Baltimore has sent a letter to each member of the state legislature, giving numerous reasons why the amendment should be passed. The following is an extract from the letter:

> It is universally recognized that mathematics, science and engineering subjects are the greatest factors in the education of the human intellect and development of the power of differentiating substance from elusive matter. An engineer is the best-prepared person to grasp intricate and complex problems in any line involving public utilities. Any commissioner, before he is of much service as such, must be educated along these lines.
>
> The problems of the Public Service Commission are legal, financial and engineering. The greater part of those thus far presented have been engineering questions, such as the gas-rate case, the telephone case, the orders issued to the United Railways and Electric Co. relative to physical valuation, etc. The cost of transportation—in fact, the cost of anything in the public-service line—is an engineering problem. The handling or operation of the service of a public-service utility is also an engineering problem.
>
> Therefore it is advisable that at least one commissioner have the training and education that will enable him to pass on engineering problems without being entirely dependent upon the statements of interested experts.

This bill, if passed, will not become a law until June 1, 1916, so it does not affect the appointment of the commissioner whose term begins May 1. As the next term does not expire until May 1, 1918, it can readily be seen that this bill is being presented from a motive of public welfare and not to further the interests of anyone in particular.

## United States Patent Office Force Reorganized

A bill for the reorganization of the working force in the United States Patent Office was signed by President Wilson on Feb. 15. This law is expected to increase the efficiency of the Patent Commissioner's staff by encouraging competent examiners to remain in the Government service instead of going into private employment at the time when their training has made them valuable.

By the office reorganization now authorized the number of examiners in the several grades is equalized, which will permit of faster promotion from the lower grades. Under the old organization there have been 110 fourth assistant examiners, 88 third assistants, 73 second assistants and 63 first assistants. Under the new plan there will be 86 examiners in each of these grades. With the old plan and the growth in the work of the office each primary examiner had to supervise the work of twice as many assistants as was the case 30 years ago.

The new law provides for increasing the number of assistants of higher grades in each division, giving the primary examiners the help of assistants better trained in their work. The number of examiners-in-chief is also increased from three to five. These men constitute the board of appeals of the Patent Office and have to pass upon about 1,100 cases per annum. Under the old law there was no way of assigning anybody else to aid in this work, even in case of protracted illness of members of the board, so that the lengthy illness or absence of two examiners-in-chief automatically put a stop to the hearing of appeals. The new law also increases the number of law examiners to five. Heretofore the law provided that two of the second assistant examiners might be women; the new law removes this limitation.

The effect of the reorganization of the office under the new law will be to increase the average salary of assistant examiners from $1,880 to $1,950 per annum. The law increases the number of positions in the office by 13 and adds $55,500 to the amount of the Patent Office payroll. Last year the office turned into the Treasury a surplus of $240,000.

The plan for this reorganization was worked out by Commissioner Ewing last year, and the bill then passed both houses, but was sent from the Senate to the House on the day of final adjournment, too late to be signed by the Speaker. This year the measure was placed near the top of the Senate calendar at the beginning of the present session and was soon passed by both houses.

## Nashville Paving Work, 1914-15

Commendation of the general character of the work, criticism of the way the contracts were let and recommendations for bettering conditions in the future are features of a report on paving contracts at Nashville, Tenn., during 1914 and 1915, aggregating $1,100,000. The report was submitted to the City Commission, or Council, on Jan. 31 by James Cameron and staff, auditors, of New York City, and Walter J. Douglas, of Barclay Parsons & Klapp, consulting engineers, New York City. The report is one of a series that may be said to constitute a broad and detailed survey of the city affairs of Nashville.

As to the character of the work the report states that the construction was of high quality, as shown both by laboratory and by field tests. A peculiar feature of the paving contracts was that bidders were required to give separate figures on construction and maintenance for a five-year period, although the contract awards were based on the construction figures only. Under this plan a number of contracts were given to bidders whose combined construction and maintenance figures were higher than the total of these figures as submitted by other bidders. On the completion of the work, payment was made both for construction and for maintenance. The report urges that hereafter, in case separate bids are taken on construction and maintenance, the commissioners postpone payment for maintenance in order to insure prompt repair of pavements at any time during the guarantee period.

In the course of a somewhat lengthy discussion of contracts for bitulithic pavements the report declares

that a license provision in the specifications for this pavement practically eliminated competition. In the opinion of the authors of the report the city might have saved 45c. a yard on 156,000 sq.yd. of bitulithic pavement laid in 1914, or a total saving of $70,000 if there had been open competition for the work.

The penetration type of bituminous pavement has been too extensively used in Nashville, according to the report, in view of the fact that "it is more or less experimental." The report recommends that no more penetration pavements be laid in Nashville until those already down or contracted for have been proved satisfactory by "at least two years of service."

Prior to 1914 Nashville had comparatively little pavement, the report states. A large amount of work was started in 1914 without systematic planning, although it is to be said for the work of that year that it was done mostly on the main arteries of travel, whereas the work in 1915 was scattered at random over the city.

Nashville has 217 mi. of paved streets, 83% of which is waterbound macadam. A large part of this waterbound macadam must be covered with some kind of bituminous wearing surface. For this purpose bituminous concrete is recommended, or bitulithic if the latter can be secured at reasonable price. The heavily traveled streets, however, should be paved with asphalt upon a concrete base or else with brick or granite.

A noteworthy recommendation in the report is to the effect that hereafter all discussion between municipal officers relating to contracts be in writing.

In view of the fact that Nashville ought to undertake a considerable amount of paving work in the next ten years the report makes the pertinent suggestion that the city engineer, acting under instructions from the commission, should prepare a complete program looking a number of years ahead, elastic enough to permit changes to meet special conditions that may arise and also sufficiently elastic to take into account the wishes of property owners. For the immediate future a paving expert should be employed. Beyond and above this, all the engineering work of the city ought to be put in charge, the report states, of a Director of Public Works possessing municipal experience and preferably a member of the American Society of Civil Engineers. A minimum salary of $4,500 and appointment for a five-year term are advised. The city should also employ chemical engineers in connection with its paving work.

## State Engineering in Oklahoma

EDITORIAL CORRESPONDENCE

If engineers in the East think their efforts and work are not appreciated, they should live in Oklahoma for a while. The office of State Engineer of Oklahoma was created in 1915. On July 1 of that year, W. P. Danford was appointed Assistant State Engineer and on Aug. 10 made Acting State Engineer with the State Engineer's salary of $2,100 per annum. However, of this munificent sum of $175 per mo. he was instructed to set aside $50 per mo. to cover incidental expenses.

These incidental expenses included the salary of an assistant at $50 per mo. to register automobile licenses (for over two months) and the payment of the incidental expenses of the professor of civil engineering of the State University, who assisted in the organization of the State Highway Department. Of this $50 a mo. but $57.27 remained unexpended on Jan. 1, 1916.

These are some of the duties performed by the State Engineer in the first six months of his term: He conducted examinations (with the help of an examining board) of 88 applicants for registration as county engineers; he had to interview between 800 and 1,200 visitors, who came mostly from a distance and on business pertaining to roads and bridges, and it was incumbent upon him to give ample time to these people to explain their local conditions and problems; he had to maintain a correspondence of 10 to 15 letters a day, and there was no stenographer except one he could borrow occasionally from the state pardon clerk, frequently two weeks between borrowings; he had to travel 3,000 mi. over the state on business trips, being gone from a day to a week at a time; and he had to perform all the routine of his office as well as the engineering work, because the assistant state engineer's time was entirely taken up with registering automobile licenses. He did not even have the funds to buy office equipment and had to file his plans and profiles in the drawer of his drafting table. The only chair in his office besides his own will not stand alone.

With all these handicaps, besides examining applicants for county engineer, the State Engineer has examined and approved 250 plans for bridges, to cost $125,000, and 50 plans for culverts, to cost about $20,000. He has examined and rejected about 50 bridge and culvert plans. He has drawn up blank forms for county-engineer reports and compiled statistics of 12,604 mi. of road that has been designated as state roads; of 994 mi. of state road surveyed; of 1,050 mi. of road built and of the money expended in the different counties on roads.

The state-highway law is indefinite and deficient in many points, and the State Engineer has little real authority. The road work, as in Kansas, is wholly in the charge of the county commissioners. The only road work in which the state may directly engage is that on which convicts are employed, and even in such cases the counties pay the transportation expenses and the cost of feeding stock, the guards' board bills, supplies, power, etc.

## To Train Soldiers by Mail

Governor Whitman has launched a project for organizing the engineers in the employ of New York State into a military reserve corps. The invitation to enroll has been extended through State Highway Commissioner Duffey, and it is expected that about 1,200 men will be available for training. It is planned to give correspondence courses in military theory and to supplement them by periods of intensive drill and practice in the regular-army instruction camps such as the one started at Plattsburg, N. Y., last summer.

## Extensive Road Improvements

A $3,000,000 bond issue for permanent road improvements in St. Louis County, Missouri, was authorized by a large popular majority on Feb. 15. The county has had a system of macadam roads, which for a number of years have been kept in good repair and oiled. These roads get hard service from the City of St. Louis.

The cost of maintenance is now so high that it is considered a measure of economy to construct more permanent road surfaces. It is estimated that the $3,000,000 voted will be enough to reconstruct all the main arteries leading out from St. Louis.

**A Rear Collision** on the New York, New Haven & Hartford R.R. at Milford, Conn., at about 11:30 a.m., Feb. 22, killed 10 persons and injured 30 others. A westbound local passenger train crashed into the rear of a westbound express stalled ahead of it, and the two trains, jumping from the tracks, were sideswiped by a freight train moving west on the adjacent track. The local was steam operated, and the boiler of its locomotive burst. The railroad company states that all signals were properly set. Survivors allege that they heard a steam whistle blown for 1½ min. just prior to the collision and believe that it could only have been sounded by the freight in an endeavor to warn the express. Examination showed the brakes on the local to be in good order and that they had been set hard. The engineer of the local was killed, as was also the flagman of the express. Most of the other dead and injured were in the rear coach (steel) of the express.

**Air Blowout in Subaqueous Tunnel Work**—A blowout occurred at 4 p.m. on Feb. 19 in the north tunnel, east end, of the South Ferry-Montague St. subway tunnels now building under the East River, New York City. Two parallel tunnels are being driven by means of shields, and the work has been pushed out under the East River a distance of about 600 ft. from the foot of Montague St., Brooklyn. The shield in the north heading was about to be moved forward its 26-in. lift and, to this end, the shoring in the excavation was being removed by two sandhogs, when a hole began to form in the roof of the excavation in front of the shield. A workman attempted to plug the hole with a bag of sand, but was drawn up and into the river. Immediately following, the two who were removing the shoring were drawn up and out. One of the latter men survived. The excavation apparently had proceeded under a pocket or depression in the mud, so that the mud roof had thinned, reducing the resistance against the 24-lb. air pressure maintained at that time. As the hole enlarged, water entered the tunnel until it was flooded. Repair work started on Feb. 21, by dumping clay over the break. A blowout occurred on Nov. 30, 1915, in the south tunnel, west end, of that other pair of tunnels which completes the East River connections between the New York and Brooklyn subways now under construction. This latter pair connects Old Slip, New York City, with Clark St., Brooklyn. The Flynn-O'Rourke Co. is contractor for both jobs (four tunnels).

**Smoke-Prevention Education**—As a part of the campaign of the New York City Department of Health, the Bureau of Public Health Education, C. F. Bolduan, M. D., Director, has issued a pamphlet for steamboat owners, describing how the firemen can save coal and keep the air clean.

**A Rock-Road Bond Issue of $1,300,000** was defeated by the voters of Saline County, Missouri, on Jan. 19 by a vote of 2,969 against to 2,310 for. A total of 192 mi. of improved roads was proposed. It is stated that the defect was chiefly due to "a nasty municipal fight in Marshall, the county seat and principal town."

**Kansas City, Mo.**—Bonds to the amount of $475,000, payable in 20 yr. at rates of 4 and 4½% annually, have been sold to St. Louis and Boston bankers at a premium of $12,680, or about 3%. Bonds to the amount of $25,000 in denominations of $100 brought bids from but two individual investors. The proceeds of the bond issue are to be used for the construction of buildings for the general hospital, the fire and police departments and the grading of the park in front of the Union station.

**The Irrigation Canal System** of the Imperial Valley, California, was bought at auction on Feb. 8 by the Southern Pacific Co. for $3,875,000. This is a step in the transfer of the property of the California Development Co. to the Imperial Irrigation District. It is expected that the transfer will be completed soon. Col. William H. Holabird is receiver, and Chester Allison is chief engineer for the California Development Co. C. R. Rockwood, El Centro, Calif., is chief engineer of the Imperial Irrigation District.

**The Can and Platform-Wagon** system of garbage collection has just been adopted for Utica, N. Y., by Commissioner of Public Safety, H. R. Hayes. Eight wagons and about 700 cans will be required by the contractor, J. R. Baxter. The house accumulations are transferred to the wagons as before; each can is cleaned and sterilized every time it goes to the reduction plant. The contractor receives $2,178 per mo. and the collection contract has just been signed for 5 yr. At present Utica has a population of about 85,000. The contractor for the reduction of the garbage is Henry Steppenbach. A fertilizer plant is maintained, the most of the grease being used as fuel and a little sold for soap. Some 50 to 60 tons a day are received, the contractor getting $4,097 per year. Ashes are collected and dumped for $19,560 per year, the contractor (Utica Carting Co.) buying property for dumps and selling it after the fills are completed.

**All the Pump-Building Plants** of the International Steam Pump Co. are to be sold under foreclosure to the highest bidder, under decree of the United States District Court for the Southern District of New York. The property is to be sold in 15 different parcels, several of which are so arranged as to include one principal works of the company. Among the principal plants to be sold are the Deane Steam Pump works at Holyoke, Mass.; the Laidlaw-Dunn-Gordon Pump works at Hamilton, Ohio; the Snow Steam Pump plant at Buffalo, N. Y., and the Power and Mining Machinery plant at Cudahy, Wis. Other parcels include stock in the Henry R. Worthington Co., the Blake & Knowles Steam Pump works, the Holly Manufacturing Co., the Fred M. Prescott Steam Pump works, the Denver Rock Drill and Machinery Co., the International Gas Engine Co., and the Jeanesville Iron Works Co. The dates of sale of the different parcels range from Feb. 25 to Mar. 7. The sale is made in accordance with the plan adopted for the reorganization of the company.

**Prizes for the Best Designs** for workmen's houses amounting to $2,100 are offered by the "Roosevelt America-First Fund." The prizes are offered in connection with the movement for better housing of newly arrived immigrants in America who are employed in connection with industrial operations. The competition is divided into two classes. Those competing in the first class should submit designs for the following: (a) Single-family house; (b) combined family and lodging house; (c) boarding house or community dwelling. The prizes offered in this class are a first prize of $1,000; second prize, $500; third, fourth and fifth prizes of $100 each. In the second class a first prize of $200 and a second prize of $100 is offered for the best design for a boarding car for use of a railroad construction gang. A public exhibition will be held of the prize-winning plans and other plans selected by the jury of awards. The contest is to close on May 1, 1916. Among the members of the committee in charge of the competition are Frank Trumbull, Chairman of the Board of Directors of the Chesapeake & Ohio R.R.; Felix M. Warburg of the Federal Reserve Board, and George A. Cullen, of the Delaware, Lackawanna & Western R.R. Further information regarding the conditions of the contest may be obtained from the "Immigrants in America Review," 20 W. 34th St., New York City.

**Engineers and State Engineering in Minnesota**—An engineering staff connected with the university-extension division of Minnesota State University, to furnish information and advice to municipalities in regard to public improvements contemplated, has been advocated by the League of Minnesota Municipalities, presumably on lines already followed in Wisconsin. Such a move would require authority and appropriation by the state legislature. It appears, however, that the sanitary division of the Minnesota State Board of Health already deals with matters of water-supply, sewerage and sewage disposal, while the State Highway Commission reports upon bridges and roads. Therefore, paving is the only important class of improvements not already covered, and most cities interested in paving employ city engineers.

These conditions were reviewed in a report presented at the annual meeting of the Minnesota Society of Surveyors and Engineers in St. Paul on Feb. 10. It was shown that the proposed engineering staff would involve a duplication of efforts, and the Society adopted a resolution opposing the creation of such a staff. It also recommended that state departments should refrain from indorsing or recommending engineers for proposed work, the information given by such departments in response to inquiries being limited to the names of municipalities where similar work has been done. This course would leave the inquirers to investigate for themselves the character of the service rendered and the qualifications of the engineers available.

## PERSONALS

Mr. Glenn P. Beach, Principal Assistant Engineer of the Minnesota Railroad and Warehouse Commission has resigned to engage in other business.

Dr. Eliot Blackwelder, Professor of Geology at the University of Wisconsin, has been appointed Head of the De-

partment of Geology at the University of Illinois. Dr. Blackwelder was paleontologist with the Carnegie Institute expedition in China in 1903. He has been a member of the Wisconsin faculty since 1904.

Mr. W. W. Cunningham, for the past 13 years President and General Manager of the Pennsylvania Clay Co., of Pittsburgh, Penn., has resigned his position and is now associated with the Clydesdale Brick and Stone Co., Oliver Building, Pittsburgh, as Vice-President. He will have charge of the manufacture and sales of the "Clydesdale" shale paving block that will shortly be put on the market.

Mr. John T. Hopkins, Assistant Engineer, of Oswego, N. Y., has been appointed Borough Engineer of Highland Park, N. J. He will virtually be borough manager; but as there is no statutory warrant for appointing a borough or city manager in New Jersey, the legal responsibility for his work other than as borough engineer will rest on the mayor and council. Besides the legal compensation for serving as borough engineer Mr. Hopkins will be paid a further sum, provided by public-spirited citizens who desire to see the manager idea given a trial. Mr. Hopkins was one of about 200 men who responded to an invitation for applications for this position which was printed in "Engineering News."

Mr. D. W. Cole, M. Am. Soc. C. E., who has been in the United States Reclamation Service for nearly 12 years, recently in charge of the Truckee-Carson project and construction of the Lahonton dam in Nevada, has been designated Senior Engineer and resident Manager of the Boise project, succeeding F. E. Weymouth, M. Am. Soc. C. E., who was recently appointed Chief of Construction for the Service at Denver, Colo. He will have general oversight of all projects in the States of Washington Oregon, Idaho, Nevada and Northern California, for the purpose of advising the Chief of Construction upon all reclamation affairs in that territory. His headquarters will be at Boise, Idaho.

Mr. P. P. Borden, M. Can. Soc. C. E., of Montreal, has been appointed a Member of the Board of Engineers of the Quebec Bridge, filling the place made vacant by the death of C. C. Schneider. Mr. Borden is a graduate of McGill University and for several years after graduation was assistant engineer in the bridge department of the Canadian Pacific Ry. From 1904 to 1906 he was Assistant Chief Engineer, structural department, Montreal works of the American Locomotive Co. From then until 1908 he was Structural Engineer of the Canadian Pacific Ry. In 1908 he became Assistant Engineer, Board of Engineers, Quebec Bridge, and for the last three years has been Assistant to the Chief Engineer, C. N. Monserrat.

## OBITUARY

Alexander Saunders, President of D. Saunders' Sons, Inc., of Yonkers, N. Y., manufacturers of pipe machinery, died at his home in that city, on Feb. 1, in his seventy-eighth year.

William Evarts Tracy, recently on the engineering staff of the Anaconda Copper Co., at Helena, Mont., and for 10 years previous to that connection Engineer of the Liberty Bell Mining Co., at Telluride, Colo., died at Helena recently, at 38 years of age. He was born in Plainfield, N. J., and was graduated from Yale University in 1900, after which he attended the Columbia School of Mines.

Louis Duncan, Past-President of the American Institute of Electrical Engineers, a member of the firm of Duncan, Young & Co., Consulting Engineers of New York City, died at his home in Pelham Manor, N. Y., on Feb. 13, at 54 years of age. Dr. Duncan was born in Washington, D. C. He was graduated from Annapolis in 1880 and three years later was sent by the United States Government to Johns Hopkins University to take a special course in electricity and physics. He received the degree of doctor of philosophy from Johns Hopkins in 1887 and resigned at that time from the navy to become a member of the Johns Hopkins University faculty. He established the course of electrical engineering of that university and was head of the department of electrical engineering until 1898. During that time he became associated with Frank J. Sprague and Cary T. Hutchinson and acted as consulting engineer on the electrification of the Baltimore tunnels of the Baltimore & Ohio R.R. and the equipment of the Third Ave. elevated-railroad system of New York City, later being consulting engineer also of the New York City Rapid Transit Commission. During the Spanish-American War he became major of the First Volunteer Engineers. In 1899 Dr. Duncan inaugurated the electrical-engineering course at the Massachusetts Institute of Technology and was head of the department of electrical engineering from 1902 to 1904. He was twice elected president of the American Institute of Electrical Engineers (1895 to 1897). Dr. Duncan was a writer of numerous articles on engineering subjects contributed to the technical press of this country.

## ENGINEERING SOCIETIES

COMING MEETINGS

AMERICAN ROAD BUILDERS' ASSOCIATION.
Feb. 28 to Mar. 3. Annual convention in Pittsburgh. Secy., E. L. Powers, 150 Nasau St., New York City.

ENGINEERING SOCIETY OF WISCONSIN.
Feb. 25-26. Annual meeting in Madison. Secy., L. S. Smith, Madison, Wis.

**Kentucky County Road Engineers' Association** held a convention in Lexington on Feb. 4 and 5. There were four formal addresses pertaining to road construction and a number of short talks.

**North Dakota Society of Engineers** is holding its eighth annual convention, Feb. 24 and 25, at Grand Forks. The program promises sessions of unusual interest. The secretary is E. F. Chandler, University.

**The Florida Drainage Association** will hold its third annual meeting at Sanford, Fla., Feb. 28 and 29. In addition to various addresses on the subject of land reclamation for agricultural and other purposes there will be an exhibition of ditching, leveeing and pumping machinery. For further information address Benjamin Getzoff, chairman of Committee of Arrangements, Bartow, Fla.

**American Building Show** has been going on in Cleveland since Feb. 16 and will continue through Feb. 26. In the first week of the show the annual convention of the National Builders' Supply Association was held. This week the National Brick Manufacturers' Association is in session, as are also the National Paving Brick Manufacturers' Association, the American Ceramic Society, the Hollow Tile Manufacturers' Association and the Ohio Association of Lumber Dealers. The daily attendance at the "First American Complete Building Show" is estimated to average 10,000.

**San Francisco Association** of members of the American Society of Civil Engineers at its eleventh annual meeting elected H. L. Haehl, president and Jerome Newman vice-president. A communication was read from Mr. Newman, suggesting that an effort be made to ascertain the views of the association as to the extent to which it is justifiable for one engineer to use the plans and specifications of another, without permission and without giving credit to the latter. A committee of five was appointed to consider and define principles of ethics in engineering practice and report at a subsequent meeting.

**A Snow-Removal Conference** will be held on the evening of Mar. 8 in New York City in the assembly hall of the Automobile Club of America, 247 West 54th St., under the joint auspices of the National Highway Association, the Automobile Club of America, the Citizens Street-Traffic Committee of Greater New York and the graduate course in highway engineering, of Columbia University. The conference will open at 8:30 with an illustrated lecture on "Modern Methods of Snow Removal" by Wm. H. Connell, of Philadelphia. The proceedings of the conference will be published by the National Highways Association, Charles Henry Davis, president.

**Boston Society of Civil Engineers** held its seventh annual dinner jointly with the Boston members of the American Society of Mechanical Engineers and the American Institute of Electrical Engineers at the Boston City Club on Feb. 8. More than 500 members of the three societies were present. Charles L. Edgar presided as toastmaster, and the speakers were Governor McCall, of Massachusetts, Col. W. E. Craighill, Corps of Engineers, U. S. A., William Barclay Parsons, John R. Freeman and Dr. John A. Brashear. Governor McCall spoke of the need of engineers in the economic and efficient administration of the government and on the desirability of reducing a large number of commissions in Massachusetts. Colonel Craighill spoke of the work of military engineers in the time of war and the plan of the War Department to form a reserve corps of engineers. He described briefly the work that this reserve corps would be called on to do for the defense of Boston. Mr. Freeman spoke of his recent trip to investigate the slides in the Panama Canal.

**Building Officials' Conference** held its second annual meeting on Feb. 14, at the Auditorium Hotel, Chicago. About 30 representatives of building departments and bureaus throughout the country were present. The morning and part of the

afternoon sessions were devoted to discussing the recent changes and improvements in building codes and the desirability of giving building officials discretionary power, a large part of that discussion being devoted to the question of the certificate of occupancy. Other topics that occupied some of the time were "The Use of Terra-Cotta Blocks and Gypsum Products" and "Metal Lath and Wood Shingles." The discussion on these matters proved so interesting that an evening session had to be arranged for in addition to those scheduled. The following important resolutions were adopted by the conference:

Resolved, That discretionary power should be vested in building officials to vary or modify the law, where practical difficulties arise in its literal enforcement, so that the intent of the law is secured; and that in case the building official declines to exercise such discretionary power, an appeal should lie to a board of technical experts.

Resolved, That proper means of egress from every building should be provided in the form of inclosed stairways; that outside metal fire escapes should not be considered as a required means of egress from new buildings, and that their use should be confined to existing buildings where other means of egress are impractical; and that wherever such fire escapes are used, the opening thereon should be protected by metal sash and wire glass or by fire-resisting doors with proper fusible links.

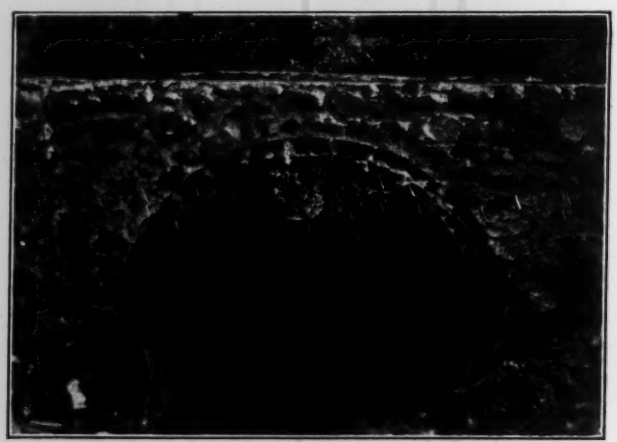
The meetings of this conference have up to this time been informal in character. It was decided, however, to form a more permanent organization. Active membership will be open to any department or bureau administering laws or ordinances affecting the safety of buildings, with respect to their construction, fire hazard, or sanitation, or to individuals actively connected with such departments or bureaus. It is intended to hold an annual meeting each winter, the meetings to be held in different cities and so far as possible in connection with some exposition or meeting of some other organization having somewhat similar interests.

Rudolph P. Miller, of New York City, was elected to serve as chairman of the conference for 1916, and James G. Houghton, Building Inspector of Minneapolis, was elected vice-chairman. Sidney J. Williams, State Building Inspector of Wisconsin, was elected secretary. Superintendent Carlin, of Brooklyn, N. Y., and Building Commissioner Forbush, of Newton, Mass., were elected members of the executive committee.

## Appliances and Materials

### Vitrified Segmental Culvert Blocks

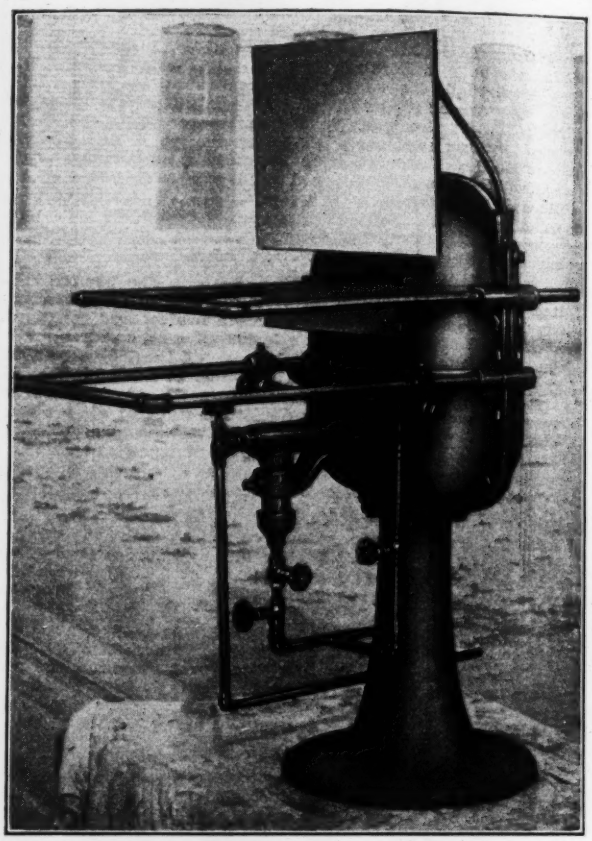
Vitrified segmental blocks, which have been quite extensively used the last two or three years in sewer construction, are being introduced for highway culverts, particularly in the clay country of the Middle West. The accompanying illustration shows an 84-in. culvert in Minnesota built of these blocks. A 24-in. by 40-ft. culvert in Kansas was built by two inexperienced men and a helper in one working day's time. The arch centers used for this work were merely three templets cut to a 24-in. radius and wedged up on 4x4-in. sticks placed across the culvert just below the springing line. The templets were covered with 1x4-in. strips. On completion of a section of the arch the wedges and centering were removed and backfilling proceded at once, without waiting for the mortar to set. Adjacent blocks are dovetailed to prevent their being displaced. These blocks are manufactured by the W. S. Dickey Clay Manufacturing Co., Kansas City, Mo.



LARGE HIGHWAY CULVERT OF SEGMENTAL BLOCKS

### Oil-Burning Furnace with Preheater

The No. 3 "Leyner" oil-burning furnace, which is used extensively for heating drill steel, is now made with a preheating chamber designed to increase the capacity of the burner. The preheater is a section that fits between the body and the cover of the furnace, and it is a simple matter to attach the section to an old furnace, the addition requiring only the changing of a few bolts. The manufacturer, Ingersoll-Rand Co., claims that by the addition of the preheating feature the space for heating is doubled, the heating capacity of the furnace is increased about 50% and the efficiency to a corresponding extent.



NEW "LEYNER" NO. 3 PREHEATING FURNACE FOR HEATING DRILL STEELS

The type of burner now furnished is suitable either for high- or low-pressure air. For high pressure it has merely to be throttled. The furnace burns petroleum or any of its oils. The floor space occupied by the No. 3 furnace is 3x4 ft., and no foundation is required.

* * *

### Dump Buckets Equipped With Wheels

Wheeled skips, or dump buckets, developed for use in removing subway spoil from the various shafts on Broadway, New York City, are manufactured by the G. L. Stuebner Iron Works, Long Island City, N. Y. Being equipped with wheels, they do not have to be conveyed by cars from the foot of the shaft to the point of loading. The bucket is designed so that it will not tip while being hoisted, although when it is desired to dump the bucket a trip line is used. The hosting handle carries two latches, one on each side, which engage with the trunnions of the skip and cannot be released until the bucket is set down.

* * *

### Weir with Flow Proportional to Head

A weir of special shape, the flow over which is directly proportional to the head on it, has been devised by H. C. Alger, of the Kennicott Co., Chicago Heights, Ill. All cams and complicated mechanisms in the recording device of a water meter using this weir are eliminated, and a simple recorder for pressure head is calibrated to read flow directly. The curve of the Alger weir was obtained by experimentation and found to be a right cycloid. A line of "Cycloid Weir Meters" is being made by the Kennicott Co.